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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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PRESENT STATUS OF SANITARY ENGINEERING; SUGGESTIONS FOR OBJECTS AND AIMS OF THE SANITARY ENGINEERING DIVISION*

By Harrison P. Eddy,† M. Am. Soc. C. E.

The field occupied to-day by the sanitary engineer is so broad that it is not easy to frame a definition of sanitary engineering which is sufficiently comprehensive to include all his activities.

According to Merriman:‡

"Civil engineering is the art of economic construction, that is, the art of making structures for the public use at the minimum cost for installation and operation."

From this basic statement, he derives the following definition:

"Sanitary engineering is that branch of civil engineering which is concerned with the constructions for promoting the health of the community."

This definition, however, is not sufficiently comprehensive to include many engineering works which are popularly conceived as coming within the field of sanitary engineering. Thus, an important function of municipal waterworks is that of furnishing adequate fire protection which can be classed as a health measure only by courtesy. Again, the treatment of industrial wastes in some cases is required merely to secure the acceptable appearance of streams, on the theory that the public is entitled to enjoy the rivers in substantially their natural condition.

Whatever may have been the early conception of the scope of sanitary engineering, its field in recent years has been greatly broadened. It is now common to link together in matters coming under the jurisdiction of health boards and departments, not only the health and comfort of the public, but also its sensibilities. The sanitary engineer is called on for assistance in measures affecting public comfort and sentiment about as frequently as in those relating directly to the public health. Many alleged nuisances formerly believed to have been detrimental to health, are now recognized to be merely inimical to comfort.

The advance in industrial sanitation has introduced many problems of an engineering nature. The manufacturer consults the sanitary engineer relative to the volume and the quality of the supply of water available for his use, the practicability of discharging the liquid wastes into the neighboring river or lake, or the type and cost of treatment plant which will be capable of rendering his wastes suitable for such discharge. Similar advice may be sought regarding offensive gases which may escape from the factory; for the manufacturer is becoming more and more jealous of the esteem in which his neighbors hold him. On the advice of the sanitary engineer may depend the

* Presented at the meeting of the Sanitary Engineering Division, January 16, 1923.

† Cons. Engr. (Metcalf & Eddy), Boston, Mass.

‡ "Elements of Sanitary Engineering", Third Edition, p. 8.

decision of the manufacturer to proceed with a contemplated project, or to abandon it for one more favorably situated.

A large proportion of the problems with which the sanitary engineer is dealing may be included in five principal divisions: Water supply and purification; sewerage, drainage, and the disposal of sewage and industrial wastes; refuse collection and disposal; eradication of the mosquito and other vermin which may carry infection; and air supply and purification.

Water Supply and Purification

The problem of obtaining and distributing an adequate supply of pure water is one of the oldest in the field of sanitary engineering. It has received more scientific investigation, followed by careful engineering operation, than any other problem of this branch of engineering. Records of such investigations and operation have afforded a vast supply of data for study and comparison.

At the time the first municipal water supplies were established in the United States, little difficulty was experienced in obtaining pure water. The municipalities were small, and the volume of water required was moderate. The neighboring country was sparsely populated, and there was little danger of contamination. Later, as the catchment areas became more populated, the possibility and, in some cases, the probability or certainty of contamination was recognized. Supplies which had been of satisfactory quality had to be abandoned for others more remote.

Some cities adopted the policy of acquiring land on the margin of the reservoirs and along the streams which fed them, thus preventing the establishment of a resident population. This was followed by police measures such as prohibition of bathing and fishing in the waters, of boating and cutting of ice on the reservoirs. Sanitary inspection followed and where the reservoir feeders flowed through populous districts, the water which they carried, was filtered before its discharge into the reservoirs.

Coincident with these improvements, valuable scientific investigations were made, which afforded much knowledge in support of the measures of protection previously taken and a basis for further progress of this kind.

The contamination of some readily accessible waters and the great difficulty of obtaining safe, acceptable supplies in some parts of this country led to the investigation and use of water filters, first, of the slow-sand type and, later, of the rapid or mechanical type.

Although the adoption of filtration may fairly be said to have been founded on the need of a water which would not endanger the public health, an important consideration has been improvement in the appearance of the water. This is particularly true in those parts of the country where the water is naturally turbid, muddy, or highly colored.

Practice in filter design and operation became comparatively well standardized, and the great accomplishments of the filter both as a means of protecting the public health and of providing an acceptable water from a very unattractive raw supply were well demonstrated. It was recognized that

filters were not 100% efficient; there were lapses, irregularities in character of output.

The next step was that of disinfection of the water, generally with chlorine, ozone and the ultra-violet ray being utilized in some of the small installations. Like filtration, sterilization has had an important influence on the problem of providing a safe water supply. In many of the smaller communities, and in some large cities, it has been the only safeguard between a contaminated supply and the consumers. Other communities have adopted the more dependable and safer course of adding this treatment to that of filtration, thus affording a valuable factor of safety.

The efficiency of disinfection has proved to be such that there has been a tendency to substitute it, in places where the water is reasonably satisfactory in appearance, for filtration or for combined filtration and disinfection. Some engineers have advocated the use of filters of moderate efficiency, with the view of depending on disinfection as the major protection against pathogenic bacteria. To what extent, other than to meet an emergency, these policies are justified, is a matter on which there is difference of opinion. This is one of the important problems before the profession at present.

Although the status of the art of water protection and purification is such that a large measure of protection against typhoid may be assured, irregularities in efficiency still occur, which even in well operated works are occasion for concern. There is also the human element which is evident in several ways, such as insufficient appropriations for proper operation, absence of moral support from higher officials, unskilled supervision, and the lapses common to operating employees. If the treatment is by disinfection alone, such deficiencies may immediately result in serving a dangerous water. Even with the best management, a very brief lapse due to the human element may jeopardize the health of a great community.

Notwithstanding the creditable present status of the art of supplying a safe and attractive water, there are many unsolved problems. Although the prevalence of water-borne typhoid has been a criterion for many years, is there reasonable assurance that other diseases are not transmitted by waters now assumed to be safe? Will not more refined methods of investigation and further knowledge disclose dangers of which little is thought at present? Under what conditions and to what extent does the alum introduced into the water pass through the filters? What are the causes of corrosion of metals and how best may they be counteracted? What will be the effect of the hydrogen-ion control of purification plants? This list might be greatly amplified. Every new discovery opens up a field for research.

Sewerage, Drainage, and the Disposal of Sewage and Industrial Wastes

Sewers have become so common in American cities that their importance as sanitary works is not always recognized. It is only necessary, however, to recall the history of sewerage in England at the time cesspools drained into basements of buildings, or to contemplate the modern theory of transmission of disease germs by the house fly, to appreciate that, as a health measure,

sewerage facilities are probably only secondary in importance to waterworks.

Those engineers who have had to do chiefly with separate sewers sometimes fail to realize that drainage, as well as sewerage, is an important sanitary measure. Except in so far as it is a means of mosquito eradication, however, there appears to be no known direct connection of a specific disease with the lack of drainage facilities. Notwithstanding this fact, lack of good drainage is deleterious to the public health. Persons who live in poorly drained localities are often forced to walk through mud and water and to live in damp quarters, which exposure has an unfavorable effect on health, even if not directly responsible for illness.

The early sewers were intended only as a means of carrying away storm water, but, later, they were used also to carry sewage. It was natural, therefore, that subsequent designs provided for carrying both storm water and sewage in the same conduits. Still later, separate sewers were proposed and built to a limited extent. Their real or apparent advantages have been considered to be so great as to lead to a change from the general practice of constructing combined sewers.

It is an interesting fact that human thought, like a pendulum, moves to extremes. Sentiment at first was strongly in favor of combined sewers; later, it swung to the other extreme favoring separate sewers and tended to discourage the construction of combined sewers. Improper use of separate sewers, however, and a re-study of the economics of the two systems indicate that the time may be approaching when the pendulum will swing in the other direction. In any case, the selection of the proper system is one of the important duties of the sanitary engineer.

Much progress in the design of combined sewers and storm-water drains is indicated by the change from the old empirical formulas to the rational method; but more dependable data are required to make the use of this method entirely satisfactory. Although rainfall data have been greatly augmented and improved during the last twenty-five years, little is known regarding the distribution of intense precipitation over such districts as are ordinarily considered in sewer design; little attention is paid to the design of inlets and catch-basins on which the flow within the sewers so largely depends; and information relative to the time of flow over the surface of the ground to sewer inlets is scarce.

One of the most uncertain problems in the use of the Kutter formula for computing the discharge of pipes and conduits to carry sewage, is the value to be ascribed to the coefficient, n. Conditions within sewers vary so greatly that extreme accuracy in this respect is not to be expected, and it is doubtful to what extent the determination of n, by measurement, will be helpful. However, it would be a source of satisfaction to engineers and doubtless would aid in the exercise of judgment, if some really good measurements could be made of flow in sewers carrying sewage.

Much experimental work has been done in sewage disposal, in which, also, the many variables and their great range of variation make it necessary that

conclusions be reached with the greatest care. The art of sewage treatment has advanced to such a state that ordinary municipal sewage can be treated successfully to almost any required extent. The more or less popular impression that the art is in the experimental stage and that, therefore, the construction of treatment plants may well be postponed, and that little progress has thus far been made, is erroneous. Sanitary engineers should take advantage of every opportunity to counteract this impression. The accomplishments in this field have been great, and much credit is due to the investigators and practicing engineers whose efforts are responsible for the advance.

With the development of each new process there is a popular inclination to look on it as a substitute for those which have gone before. Experience has shown, however, that, in general, each problem presents its own peculiar conditions and that there is a field of usefulness for every process which has proved to be of practical merit.

The most vital difficulty in the field of sewage disposal lies in securing efficient operation. As a rule, sewage is not treated for the benefit of the citizens producing it, but to protect those living in neighboring communities, who are neither citizens nor taxpayers of the city in question. It is unpopular to appropriate funds which are to be expended for the benefit of those living outside the community. The value of operation records and analytical tests is not often appreciated by city officials and too seldom by engineers.

The treatment of industrial wastes in many respects resembles that of municipal sewage. Most of the processes applicable to the latter are useful in the former. Many industrial waste problems, however, are much more complicated and difficult than the ordinary sewage disposal problem.

This is a comparatively new field of activity for the sanitary engineer, and the opportunity for research and accumulation of knowledge is great.

Refuse Collection and Disposal

The accumulation in the street of all kinds of refuse is a menace to the public health. Such refuse may contain pathogenic organisms which, when blown into the air and inhaled, may cause infection. Street cleaning, therefore, is properly classed as a branch of sanitary engineering, although it has to do more largely with operation than with the construction of works. The change from horse-drawn to motor-driven vehicles and the enormous increase in such traffic, with the resultant increase in the use of smooth pavements, have greatly modified the problem of street cleaning.

The practice of periodically sweeping streets and gutters, and of sprinkling and sweeping the pavements, has been replaced in a measure by street flushing, which is by far the most sanitary of these processes. Paved streets can be thoroughly cleaned and can usually be kept in good sanitary condition by flushing. Street flushing, however, involves a number of serious problems, some of which are dependent on the manner in which the water is applied. If the water is discharged under high pressure and impinges directly on the surface of the pavement, it may have a disintegrating effect

under some conditions. Some engineers object to the use of water upon the theory that directly or indirectly it is injurious to the pavement.

In some cases, street flushing is objectionable because of the material washed into catch-basins and sewers. The quantity of such matter also increases the burden on treatment plants. Some engineers have advocated a combination of sprinkling, sweeping, and pick-up, with flushing; and much thought has been given to vacuum cleaning which perhaps comes more nearly to being a theoretically ideal method of street cleaning.

The relation between the collection and disposal of rubbish and the public health is possibly not as clear as it is with street cleaning. There can be no doubt, however, that regular and frequent collection of rubbish aids the householder in the maintenance of sanitary conditions about his premises. By far the most difficult branch of the problem of refuse collection and disposal is that pertaining to garbage. This is true, primarily because of the unstable character of the organic matter of which garbage is chiefly composed, which becomes offensive if not disposed of promptly. Difficulties have been overcome in a measure, at least, in many cities, by providing moderately frequent collections and by improved facilities for transportation. Yet the systems and equipment used in many places are far from ideal.

Within the last twenty-five years, marked advance has been made in disposal. Engineering and scientific principles have supplanted the rule-of-thumb methods of a generation ago. It is possible to-day to prescribe the essential requirements for the successful disposal of garbage in a sanitary manner, whether it is done by burial, incineration, reduction, or feeding to hogs.

The most difficult engineering problem, and one which is connected with practically all methods of garbage disposal, has been the control of objectionable odors to such an extent as to prevent complaint. The control of dust about incinerators and of flies and rats about garbage fills and hog ranches, have also proved to be important problems.

Garbage disposal, like that of sewage, suffers from lack of interest on the part of citizens and officials. Liberal appropriations are not generally forthcoming, either for construction or for operation. Garbage collection and disposal involves operating expenses which must be defrayed directly from the tax receipts. This accounts for much of the difficulty in the solution of this problem.

This field offers great opportunity for scientific research as well as for the creation of a public attitude which will require that ample funds be provided for the successful accomplishment of the object.

Eradication of the Mosquito and Other Vermin

For many years it has been held that disease may be transmitted by means of insects and other vermin. Positive proof, however, is of comparatively recent date, as in the noteworthy case of the transmission of yellow fever by the mosquito, proved by Dr. Jesse W. Lazear, Acting Assistant Surgeon, U. S. Army, who suffered himself to be bitten by an infected mos-

quito for the purpose of demonstrating his theory. No greater sacrifice for sanitary science can be made than that by Dr. Lazear. The eradication of yellow fever and malaria from Havana, Cuba, by the late Maj.-Gen. William C. Gorgas was an achievement which demonstrated the value of the application of sanitary science for the benefit of man, and his work at Panama was even a more spectacular example.

There have been many other similar demonstrations since these. It is stated that, during the World War, 2 000 000 American soldiers stationed in various cantonments in the malarious sections of the South, were protected so effectively against malaria by sanitary engineering works that not more than forty cases of that disease occurred among the troops. There is need, however, for further research in this field. Cases are reported where large drainage projects have actually increased, rather than decreased, the prevalence of malaria. Many engineering structures, such as railroads and improved highways, have been built in a manner to create breeding places for mosquitoes.

Another important branch of this field, occupied by sanitary engineers, is rat-proofing, which is being conducted in some seaports by types of building construction which avoid refuges for rats and prevent their entrance into warehouses and other places where food is stored.

Air Supply and Purification

The need of a supply of pure air has been recognized for more than two centuries. Many problems relating to air supply lie properly in the field of the architect, the physicist, or the heating and ventilating engineer. The sanitary engineer, however, is being called on more and more to solve problems of air supply, particularly those relating to its quality. Many of these problems appear to relate to the public comfort rather than the public health.

The suppression and control of objectionable odors has assumed considerable importance in the design of sanitary engineering structures, such as garbage and refuse disposal plants, sewers, catch-basins, and sewage treatment plants. In fact, the principal object of many sewage treatment plants has been to prevent the objectionable odors which have emanated from polluted streams. Similar problems arise in many industrial plants such as abattoirs, rendering works, fertilizer factories, and oil refineries. These problems naturally fall within the field of the sanitary engineer.

From the assertions of seventy-five years ago, that sewer gas was a deadly miasma laden with elements of almost every known disease, and of Charles Murchison* that typhoid fever was "produced by emanations from decaying organic matter", the accumulation of knowledge has led to the present conception that "the chance of direct bacterial infection through the air of drains and sewers is so slight as to be practically negligible".† The former theory, however, erroneous as it may have been, was not without its beneficial effects, for it undoubtedly resulted in many sanitary improvements.

* "The Sanitary Significance of Bacteria in the Air of Drains and Sewers", C-E. A. Winslow, Mass. Inst. Tech., *Sanitary Research*, Vol. V, p. 39.

† *Loc. cit.*, p. 85.

Although present knowledge indicates that sewer air is comparatively free from pathogenic organisms and poisonous gases, it is important that familiarity shall not breed contempt, for danger may lurk in the air within sewers. Under some conditions, products of putrefaction, like methane and carbon dioxide, may prevail to such an extent that the air will not support respiration. Large quantities of hydrogen sulphide may be present, as at the Los Angeles Siphon;* illuminating gas may also occasionally be found, and since the development of the modern garage, gasolene has become an almost constant menace in the sewers of large cities. Odors from foul deposits, septic sewage, and industrial wastes may escape from sewers through defective plumbing, manholes, and gutter inlets. Remedies for many such conditions are known, but there are unsolved general and local problems which will make their demands on the ingenuity of the sanitary engineer of the future.

Objectionable odors about sewage treatment plants have been the cause of complaint and some litigation. Present knowledge enables the sanitary engineer to accomplish much toward their elimination or control. The work of Dr. W. W. Horrocks† indicates that where sewage falls vertically the neighboring air may contain the *Bacillus Coli* and various *Streptococci*. He also found that if such an easily recognizable organism as *B. prodisiosus* is introduced into sewage, it may be recovered from sewer air into which he assumed that it had entered by the bursting of bubbles of gas rising through the sewage, from the splashing of falling sewage, or from the drying of the sewage left on the walls of sewers when the depth of flow decreased. The results of such tests led to interesting speculations regarding the bacterial flora of the atmosphere in the vicinity of some of the modern sewage treatment plants, such as fine screens, septic tanks, trickling filters, and aeration tanks of activated sludge plants.

Probably offensive odors from garbage, both during collection and disposal, have caused more general complaint than those from sewage treatment plants or industrial plants. Dust-laden air has also been a serious annoyance in some cases. Similar complaints of odor and dust from many industries have become common, have demanded recognition by State and municipal officials, and have been the subject of legal action.

The treatment of air has become as common as the treatment of water and sewage. Air is humidified and its temperature is regulated in order to make it more agreeable and, perhaps, more healthful to the consumer. It is washed and subjected to intense heat to remove dust, soluble gases, and odors. For the same reason, it is treated with oxiding agents, like chlorine and sulphurous acid, and it may be treated with deodorants like phenol, creosote oil, and pine tar oil; in a mixture of odors, if one is much stronger than the others, it will completely mask the weaker odors. This last method is particularly worthy of further investigation.

* *Engineering Record*, August 28, 1909, p. 252; "American Sewerage Practice", Vol. I, p. 551–552.

† *Proceedings*, Royal Soc., Lond., Series B, Vol. LXXIX (1907), p. 255.

Objects and Aims of the Sanitary Engineering Division

There has been a growing feeling among sanitary engineers that the Society should take an active part in the development of the art of sanitary engineering. This feeling has culminated in the formation of the Sanitary Engineering Division. Sanitary science, or the knowledge of the fundamental truths of Nature, affecting the health of man, is the foundation on which rests the art of sanitary engineering, or the application of such truths to sanitary engineering works for the health and comfort of the community.

Advancement of knowledge may come through original research and the interchange of information acquired through experience. These considerations suggest that the purpose of this Division may be defined as follows:

"The objects of this Division shall be the increase of knowledge in and the advancement of the sanitary branch of the engineering profession and the encouragement of social intercourse among sanitary engineers and sanitarians, to the end that sanitary engineers may be of greater service to the community."

In beginning the work of this Division, consideration should be given to the fact that other Societies are functioning in the field of sanitary engineering, among which are the American Society for Municipal Improvements, the American Public Health Association, and the American Water Works Association.

Duplication of effort is a waste of energy. It should be an aim of this Division not to duplicate the work done by other Societies, but to deal with underlying principles and with the state of the art rather than with mere descriptions of their practical application.

It should be an aim of this Division to record, through papers and discussions, accurate records and conclusions drawn from dependable data. Theories which appear to be sound and are predicated on known facts, may serve a useful purpose. A guide which may be helpful in determining the character of papers and discussions to be formally presented, may be found in one of the current rules regarding papers presented to the Society, as follows:

"Papers containing matter readily found elsewhere, those specially advocating personal interests, those carelessly prepared or controverting established facts, and those purely speculative or foreign to the purposes of the Society, shall be rejected."

It is a lamentable fact that many sewage treatment plants have become seriously impaired through the action of natural agencies within a few years of the date of their completion. It should be recognized that, although good engineering requires the accomplishment of the purpose at a minimum of cost, the measurement of cost must include, in addition to that of construction, the expense of maintenance and operation. It should be an aim of this Division to encourage a high grade of engineering construction. It should also encourage the highest practicable standard of maintenance.

Perhaps the most serious difficulty in sanitary engineering is that of obtaining proper operation of sanitary engineering works. Many city offi-

cials believe that operation is not a function of the engineer and that after works have been constructed they should be turned over to operating executives. Many of these executives have neither the training nor the skill to exercise efficiently the duties involved by these works. On the contrary, the operation of such works should be under the direction of the sanitary engineer, for efficiency in operation depends on the application of knowledge of sanitary science and of the principles of sanitary engineering.

It should be an aim of this Division to render assistance through co-operation with duly constituted official boards and by any other proper means, in the establishment of suitable methods and standards of operation. The need of broader knowledge is apparent in every sub-division of sanitary engineering. It should be an aim of this Division to foster research through its committees and wherever practicable by co-operating with the operators of existing plants and with other agencies organized for this purpose.

Finally, the most important function of this Division will be that of obtaining a free interchange among its members, of information developed by investigation and experience. It is a creditable fact that the engineers of this country are in the habit of cordially exchanging information and rendering assistance to one another. It should be an aim of this Division in every manner possible to foster this spirit of co-operation and to secure high ethical standards, in order best to serve the community. Such service will advance the standing of the profession.

BEACH EROSION: ITS CAUSES AND CURE

By Henry Clay Ripley,* M. Am. Soc. C. E.

Synopsis

The object of the paper is to explain specifically the principal causes of beach erosion and accretion and to determine the principles which control these actions, and thus to pave the way for the suggestion of a remedy which shall be both scientific and practical. It is believed this object has been attained. Finally, two examples are given, one in France and the other in Brazil, where works have been constructed, that illustrate fully the principles herein enunciated and are a confirmation of the conclusion reached.

There are few problems in engineering that have been fraught with greater disappointments than the efforts to prevent the erosion of beaches. Many of the devices used not only have failed to prevent, but actually have increased, the difficulty.

In order to obtain a clear understanding of the difficulties to be encountered, it is necessary to consider the action of the forces that must be controlled to obtain satisfactory results. These forces are waves and currents. The waves produce the disturbance that places the material in suspension, and the currents transport it in the direction of the flow. Without the action of the waves, the littoral currents rarely if ever would disturb the sand on the beach by virtue of their own velocity. The waves alone, however, striking the shore at an angle, will produce some lateral movement of the material, but this effect is small compared with the combined activity of wave and current. On the other hand, waves striking the beach at an angle normal to the shore line will have no tendency to produce erosion, but rather to produce accretion, as will be seen by studying the changes along a sandy coast.

Wave Action

In deep water, wave action is simply one of vertical oscillation, the amplitude of which will vary with the magnitude of the disturbing cause. When the wave reaches water which is so shallow that the depth is less than the amplitude of the wave, it assumes a motion of translation. The bottom is disturbed, and the sand is driven shoreward. The retarding effect of the bottom friction causes the surface water to gain on the bottom filaments and to heap up; the wave increases in height and, finally, breaks, depositing the sand pushed along by it in the form of a ridge, that is, a bar. The wave then

Note.—Written discussion on this paper, which will not be presented at any meeting of the Society, will be closed with the **March, 1924**, *Proceedings*. When finally closed, the paper, with discussion, will be published in *Transactions*.

* Cons. Engr., Detroit, Mich.

continues as one of oscillation, but smaller in amplitude, until for a second time it reaches a depth which disturbs its oscillatory character, when it again becomes a wave of translation. Thus, the phenomena previously described are repeated over and over again until the shore is reached. After the last break of the wave near the beach, the water is driven up the slope with considerable velocity, carrying with it the sand held in suspension, a part of which is left on the beach and a part is drawn back by the receding water. Succeeding waves repeat this process; those of the same amplitude increase the bulk of the little bars previously formed, whereas those of a different amplitude form new bars and dissipate existing ones. Thus, after every storm these little bars are found to have changed in their distances from the shore.

In the case of waves approaching the shore at an angle, the end which reaches shallow water first is retarded and the other end gains on it, so that by the time it reaches the shore its direction will be practically normal. As a result, the actual movement of sand near the shore, due to wave action alone, is practically nil. In deeper water, however, where the wave first feels the effect of depths less than its amplitude, the sand movement along the shore may be considerable.

Current Action

The current is the important factor in beach erosion. Although the velocity may be so feeble as to be absolutely useless for moving sand from a state of rest, yet when the sand is in suspension because of wave action a considerable movement takes place. Where the beach is continuous, without inlet or artificial works, this movement will be constant whenever there is sufficient current and wave action to put the sand in suspension. No actual erosion takes place because the material removed is immediately replaced by a similar quantity moving along the shore. If, however, there is a break in the continuity of the shore, as at a tidal entrance, a river mouth, or an artificial obstruction, which interrupts the continuous movement of the sand along the shore, erosion may occur. Where this interruption is due to a tidal entrance or river mouth, the moving material is deposited on the weather side of the entrance, and erosion is caused on the lee side. ("Weather" and "lee" as used in this paper relate to current movement, not to the wind.)

When the littoral current reverses its direction, the weather and lee sides change places with the corresponding accretion and erosion, and if these alternate directions and forces are equal, no actual progressive erosion or accretion takes place. Generally, however, there is a prevailing direction to the littoral current, although interrupted frequently by reverse currents due to local causes; in this case, erosion and accretion become very marked in a term of years.

When the interruption to sand movement along the shore is due to an artificial obstruction, such as a jetty projecting from the shore, several phenomena may occur, depending on the character of the structure. If the structure is low so that it is submerged at high tide, and if it is protected at the inner end so that its connection with the shore is safe from scour at

that point, there will be accretion on the weather side, which will reach the top of the work and extend nearly to its outer end. On the lee side, there will be erosion, and this will be increased by the overfall which will excavate a trench throughout the entire length of the work. There will also be erosion at the outer end due to the concentration of the current at that point. The erosion is thus somewhat greater than the accretion, and if the direction of the littoral current changes, a reversal of these effects will follow and the net result of such an artificial work will be erosion. If the work is constructed at a higher level throughout, that is, above extreme high tides and ordinary storm tides, there will be accretion on the weather side, erosion on the lee side, and scour at the outer end. An eddy will be produced on the lee side causing accretion near the shore end and erosion farther away. A succession of similar structures, spaced sufficiently close together to neutralize the eddy, will cause accretion within the area embraced as long as the high water does not overtop the works. When, however, some great storm or cyclone occurs, such as occasionally happens on the Atlantic and Gulf Coasts, causing the water to rise high above the protecting works, all accretion that may have accumulated during months or years, may disappear and there may be additional erosion aggravated by the works themselves. The net result will be to leave the beach in worse condition than it was before the works were constructed.

Another cause of erosion, which is of far-reaching effect, and not easily detected, is that due to the presence of an artificial construction extending far out from the shore, such as a long jetty or the works of an artificial harbor. On the lee side of such a construction, an eddy is formed, the effect of which may be felt a considerable distance from the work itself.

The operation of this eddy in producing erosion is as follows: The littoral current flowing past the outer end of such a structure drags the water out from the lee side, producing a current seaward along the structure. The lowering of the water near the shore causes a current along the shore in a direction opposite the littoral current. The depression thus formed causes in turn a current from the deep water toward the shore, and thus a rotary motion of the water or eddy is established. Beyond the eddy, the current resumes the direction of the normal littoral movement. Thus, there occurs within the area affected by this disturbance, a current in opposite directions along the shore, with an onshore current to supply the depression thus formed. This onshore current reaches the beach practically clear of sediment, with full capacity for transporting sand put in suspension by wave action. The erosion, therefore, is constant and cumulative as long as the littoral current continues in the same direction.

The Remedy

The causes of erosion being clearly understood, the remedy naturally suggests itself. Anything that will diminish the current or wave action will reduce or prevent erosion. In the past, efforts have been directed mainly to the reduction of current action. Frequently, this method has resulted in failure and, at other times, in actually increasing the erosive effect. It is proposed, therefore, to attack the problem from a different angle, that is, by

allowing the current to have its full sway, but controlling the wave action. This can be accomplished by building a breakwater parallel with the shore line and some distance out. The top of this structure will be below the water surface, its purpose being to reduce the height of the waves between it and the shore. This reduced wave action will cause less disturbance of the bottom and, therefore, less movement of sand within the area thus protected than on either side. Consequently, more sand will be brought into this area than will pass through it, and accretion will result. This effect will be the same no matter in which direction the littoral current flows. Even during great storms or cyclones, there will always be less scour within this protected area than outside of it, and the breakwater itself, being parallel to the shore, will impose no condition to cause erosion. This method of protection leaves the beach unobstructed, so that its use as a public drive or for bathing purposes is unimpaired.

As regards the details of the breakwater, it is difficult to lay down hard and fast rules for all conditions. Any form that will reduce the size of the waves and withstand their action, will be satisfactory. The distance from the shore at which this breakwater should be placed, will depend on the slope of the beach and other local conditions; but the structure should always be outside the low-water shore line. The elevation of its top will be governed by the range of the tide, but, in no case, should it be such as to cut off wave action altogether; for in such an event the entrance would be closed and distribution of the accretion could not be effected. The higher it is, the more rapid will be the accretion behind it. With the partial closing of the entrance by accretion, an increase in the velocity of current will be developed, which will compensate in a measure for the reduced wave action, and a more or less even distribution of the accretion throughout the protected area will take place.

A remarkable example of such beach protection is found at the Anse des Huttes Pointe de Grave near the mouth of the Gironde River, in France, where a breakwater was built parallel to the shore line and 150 m. from it. It has a length of 1 300 m., and is completely connected with the shore at one end and partly so at the other. It also has two small openings in front. Its top is about 1½ m. below high water and 4 m. above low-water level. The tidal range from low water to ordinary high water at the Equinox is 5½ m., and to extraordinary high water at the Equinox, 6 m.

In 1881, Lieut. W. H. Bixby (now Brig.-Gen., U. S. A., *Retired,* M. Am. Soc. C. E.), made an inspection of the various works for the protection of the beach at this point, and, in his report*, he states that:

"Of all these constructions the latter [the breakwater at the Anse des Huttes] has proved the greatest success, and is a grand victory gained over the encroachments of the sea. This breakwater now requires no particular attention as the beach has so widened and the sea so retreated before it that the casual visitor might wonder why it was ever built. Its success is due to its solidity and to the fact that its style of construction is such that more sand and gravel is deposited in its rear during fine weather than can be carried away even by several successive storms."

* Filed in the U. S. Engineers' Library, Washington Barracks, Washington, D. C.

If Lieut. Bixby had understood the principle involved in this style of construction, he would probably have realized that the tendency to deposit sand and gravel in the rear of the breakwater existed during storms as well as during fine weather.

This form of construction was used only after all other methods had failed. The numerous massive masonry projections rising from the broad ocean slope of the structure break up the continuity of incoming storm waves. As far as is known, this is the only case where the design and execution have fully met the conditions for beach protection as presented; it is believed that if the principles involved had been appreciated thoroughly, the cost of the structure could have been greatly reduced.

Another example is furnished by a work executed at Ceará, Brazil. This was the construction of an outer harbor to enable ships to find shelter where they could load and discharge cargo and passengers during all conditions of weather. This work was designed in 1875 by the eminent English engineer, the late Sir John Hawkshaw, Hon. M. Am. Soc. C. E. Fig. 1 shows his plan with conditions as they existed at that time.

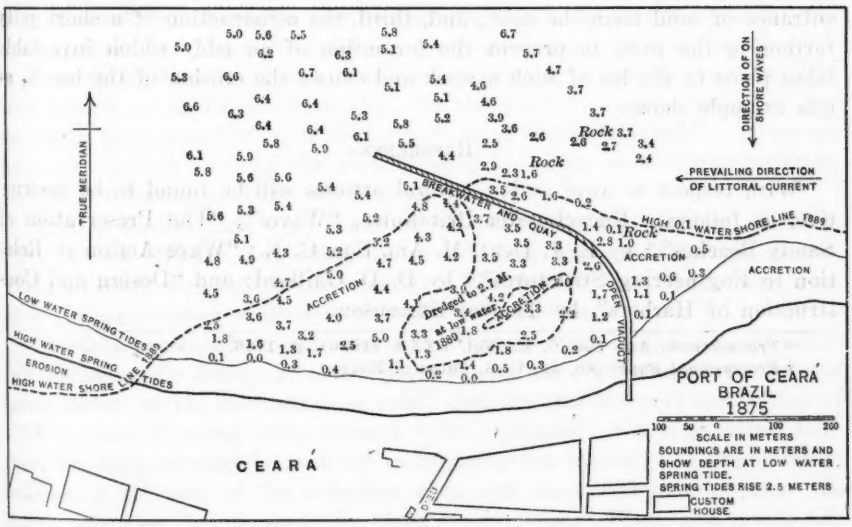

FIG. 1.

The plan consists of a solid breakwater and quay of concrete having a length of 430 m. (1 410 ft.) approximately parallel with the shore line and about 400 m. (1 312 ft.) away from it. Connecting it with the shore is an open viaduct, 250 m. (820 ft.) long, on screw-piles. To this map, the high-water shore line of 1889, taken from Chart No. 1163 Hydrographic Office, U. S. Navy, has been added. Thus, it will be seen that this artificial harbor, constructed at an estimated cost of more than $1 000 000, has been destroyed practically by the accumulation of sand. A small basin within the protected area was dredged to a depth of 2.74 m. (9 ft.). However, when the writer visited that port in 1904, the harbor had been abandoned, as it was found to

be impracticable to maintain it by dredging. A new breakwater is being constructed farther out in a depth of 4.75 fathoms (9.35 m.). The waves from the north beat directly on the shore. The prevailing littoral current is westerly and, therefore, the accretion in the harbor and to the east of it, and the erosion at the west, are strictly in accord with the principles enunciated by the writer.

As beach protection, however, the case is illustrative. The waves from the north break on the shore and thus put the sand in suspension, after which it is carried along by the littoral current to the west. When it passes through the open viaduct into the harbor, the wave action ceases on account of the protection of the breakwater in front, and the sand, therefore, is deposited in its lee. The clear water then flows out of the harbor to join the current of the eddy that causes the erosion to the leeward.

Incidentally, it may be confidently stated that the filling of this artificial harbor could have been prevented: First, by closing the viaduct to prevent the entrance of sand from the east; second, by constructing a spur from the shore toward the west end of the breakwater, leaving only an opening sufficient for the passage of vessels in and out of the harbor, and preventing the entrance of sand from the west; and, third, the construction of a short jetty farther to the west, to prevent the formation of an eddy which invariably takes place in the lee of such a work and causes the erosion of the beach, as this example shows.

References

With respect to wave action, several articles will be found to be instructive, as follows: Encyclopedia Britannica, "Waves"; "The Preservation of Sandy Beaches",* by E. J. Dent, M. Am. Soc. C. E.; "Wave Action in Relation to Engineering Structures",† by D. D. Gaillard; and "Design and Construction of Harbors", by Thomas Stevenson.

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1786.
† *Professional Paper No. 31*, U. S. Corps of Engrs.

THE DISTRIBUTION OF INTENSE RAINFALL AND SOME OTHER FACTORS IN THE DESIGN OF STORM-WATER DRAINS

By Frank A. Marston,* M. Am. Soc. C. E.

To be Presented January 17, 1924

Synopsis

The object of this paper is to present data showing the area covered by excessive rainfall of short duration based on measurements made in the Boston Metropolitan District, at New Orleans, La., and the results of one observation at Cambridge, Ohio; also to call attention to the paucity of data relating to certain other factors entering into the design of storm-water drains by the so-called "Rational Method", and to stimulate interest in these matters in the hope that additional studies and measurements may be undertaken that will make it possible to draw more definite conclusions than are now feasible.

Records from twelve recording rain gauges in the Boston Metropolitan District have been collected and studied for the years 1918-1922, inclusive, and data have been compiled from twenty periods of excessive rainfall. Diagrams have been plotted showing the areas covered for given rates of precipitation, periods of duration, and degrees of frequency.

Similar studies have been made of six periods in the storm of April 9, 1920, at New Orleans, La., using the records from six recording rain gauges. The results of one observation reported at Cambridge, Ohio, have been compiled for comparative study with the other information.

The data presented indicate that for tributary areas of the size ordinarily considered in the design of storm-water drains, the variation in the rate of precipitation within the area is so small that the fundamental assumption of uniform rate of precipitation appears to be justifiable. For large areas, however, the designer may be justified in reducing the intensity of precipitation because of the size of the tributary area, and, in addition, may make the usual reduction for increase in time of duration. These statements are tentative conclusions only and apply to the Boston area or other comparable districts. Further measurements are needed before final conclusions can be drawn.

The studies indicate that data collected in New Orleans may be applied to the Boston District by altering the estimated frequency of the intense rate of precipitation to conform to the natural conditions in the latter place. In the

Note.—This paper will be presented at the meeting of the Sanitary Engineering Division to be held January 17, 1924, during the Annual Meeting of the Society. Correspondence is invited and may be sent by mail to the Secretary for presentation at that meeting.

* Cons. Engr. (Metcalf and Eddy), Boston, Mass.

future, if this tentative conclusion can be substantiated by sufficient data, the practical value of the accumulated information will be greatly increased, and its applicability extended so as to be serviceable in other areas for which actual measurements may not be available.

An endeavor has been made to present the facts in a manner that will permit other investigators to draw independent conclusions, or to combine these records with others that may be accumulated later.

It is of interest to the profession to have additional studies made, that will produce evidence on which definite conclusions can be based regarding the distribution of intense rainfall, the run-off from small sewered areas of various degrees of imperviousness, and the so-called inlet time.

The Rational Method of Design for Storm-Water Drains

Mature judgment based on broad experience will always be of fundamental importance in the solution of storm-water drainage problems. Recently, the gathering of information on the intensity of rainfall and the frequency of occurrence of rains of high intensity has led to the development of the so-called "Rational Method" of storm-water drain design, which permits the use of available data as a guide to judgment.

Some of the principal steps in the Rational Method are:

1.—Selection of a curve giving rate of rainfall for various periods of duration for which provision should be made.

2.—Determination of the coefficient of run-off; that is, the ratio between the maximum rate of run-off and the rate of rainfall.

3.—Determination of the coefficient of imperviousness of the district; that is, the ratio of the estimated equivalent impervious surface to the total area of the district.

4.—Determination of the area tributary to the drain at each successive point for which computations are to be made.

5.—Computation of the time of concentration, or time required for water to flow from the most distant point of the tributary area to each point on the drain.

6.—Estimates of the inlet time, or average time required for water to flow over the surface of the tributary area to the nearest inlet on the drain.

7.—Computation of the rate of run-off from a given area tributary to each point on the drain.

8.—Computation of the required capacity of the drain at each point.

Intensity of Rainfall.—Considerable information is available for various parts of the United States on the probable intensity of rainfall based on records of recording rain gauges, and covering fairly long periods. Curves have been deduced from many of these records, showing the probable intensity of rainfall and frequency of occurrence of storms of a given intensity. These data are sufficiently complete to serve as a reliable basis for the preparation of rain-

fall intensity curves,* showing the relation between intensity of rainfall and duration for any given frequency of occurrence.

Coefficient of Run-Off.—Data on the maximum rate of run-off from different classes of areas are rather meager, but some measurements are available† on which to base an estimate of the ratio between the maximum rate of run-off and the rate of rainfall if the physical characteristics of the districts are known or can be estimated.

Coefficient of Imperviousness.—The coefficient of imperviousness, that is, the ratio of the estimated equivalent impervious area to the total area of the district, is made from an estimate of the area covered by pavements, roofs, sidewalks, and other impervious coverings, and a part of the area not so covered, so as to take into account the degree to which the soil is more or less impervious. For a built-up district, the area of impervious surfaces can be determined with reasonable accuracy from maps based on field surveys, or can be estimated by comparison with similar districts for which the relative area is known. For districts that are not fully developed, the coefficient of imperviousness can be estimated by comparison with the data from built-up districts.

Tributary Area.—The area tributary to the drain at each successive point for which computations are to be made, can be measured from a good map based on field surveys.

Time of Concentration.—The time of concentration, or time required for water to flow from the most distant point of a tributary area to each point on the drain for which computations are to be made, can be computed with sufficient accuracy by approximating the average velocity and corresponding time of flow in the drain, and adding the inlet time.

Inlet Time.—The determination of the inlet time, or that required for water to flow over the surface of the tributary area to the nearest inlet of the drain, is usually a question of judgment. There are only a few measurements on record to serve as a basis for judgment.

Rate of Run-Off.—The rate of run-off from a given area tributary to each point on the drain for which computations are made, is obtained by applying the coefficient of run-off and the coefficient of imperviousness to the rate of rainfall for the time of concentration, and is a matter of computation usually aided by diagrams. The Rational Method assumes that the maximum rate of run-off will be produced by a rain of uniform intensity distributed over the entire drainage area and having the same duration as the time of concentration.

Required Capacity of Drain.—The product of the rate of run-off and the area tributary is the required capacity of the drain at the point under consideration.

* "American Sewerage Practice", Metcalf and Eddy, Vol. I; "Elements of Hydrology", Meyer; "Hydrology", Mead; and, *Municipal and County Engineering*, R. E. Horton, M. Am. Soc. C. E., July, 1919, pp. 3–9.

† "Run-off from Sewered Areas", Report of Committee, *Journal*, Boston Soc. of Civ. Engrs., June, 1914, pp. 291–382; Report of Committee on Charles River Dam, 1903, diagram opposite p. 174; Annual Report, Dept. of Public Works, Portland, Ore., 1916, p. 74; Progress Report of Committee on Rainfall and Run-off, *The Municipal Engineers Journal*, N. Y., First Quarterly Issue, Vol. 8 (1922), p. 22; Discussion by Kenneth Allen, M. Am. Soc. C. E., of paper by John C. Riedel, M. Am. Soc. C. E., *The Municipal Engineers Journal*, First Quarterly Issue, Vol. 8 (1922), pp. 130–145.

The steps outlined indicate the principal points in the Rational Method that require judgment for their determination, and the extent to which available data can be utilized to aid judgment. By this method, more reliable results can generally be obtained than is possible if judgment must be used to determine factors that are less tangible and affected by many varying and more or less indeterminate conditions.

The coefficient of run-off is probably the most difficult factor to determine, because it is not constant for a given area, and is affected by many conditions. The coefficient is seldom unity, even for a surface entirely covered by a pavement. Evaporation takes place to a slight extent during rain storms, and storage and absorption occur with so-called impervious surfaces. As a rain storm progresses, the coefficient of run-off usually increases due to a decreased rate of absorption by the soil and other surfaces, and the filling of storage areas and depressions. With sandy areas, the increase is very slow in comparison with that for clayey or other more impervious areas.

The intense rain which causes heavy run-off usually occurs in the early part of the storm, and in storms of the thunderstorm type during the summer months.* Accordingly, the coefficient of run-off will generally be less than that from storms in which the intense precipitation occurs after a considerable period of rain, or at other seasons of the year.

The firm of Metcalf and Eddy has recently made allowance for these considerations by reducing the coefficient of run-off according to the so-called "zone principle".†

Information Lacking in Regard to the Distribution of Intense Rainfall

Data on the intensity of rainfall are taken from the records of recording rain gauges. For most localities a single gauge is used, thus, the records show the intensity of rain as recorded at one point, and not the average intensity for a given time over a considerable area. Where two or more recording rain gauges have been placed 1 to 3 miles apart, or even less, it has been observed that, with intense storms, there is considerable difference in the rate of rainfall recorded by adjacent gauges. The area on which rain falls at a high rate for periods of an hour or less is comparatively small. Therefore, if large areas are involved, there is the possibility of a considerable error in assuming the intensity of rainfall to be uniform over the tributary area (assumed in the Rational Method). For small areas, the effect is much less important, and may be negligible.

Practically no detailed information has been published in regard to the areas that may be expected to receive high rates of precipitation for short periods. Only a few cities have more than two recording rain gauges. The need of data on the distribution of intense rainfall has been recognized for some time. In 1911, Edgar S. Dorr, M. Am. Soc. C. E., then Chief Engineer of the Sewer Service, Department of Public Works, Boston, Mass.,

* These facts are illustrated in a paper by Robert E. Horton, M. Am. Soc. C. E., in *Municipal and County Engineering*, July, 1919, pp. 3–9.

† Described in Report to Commissioners of Sewerage upon the Sewerage and Drainage of The City of Louisville, Kentucky, by J. B. F. Breed, Chief Engineer, and Metcalf and Eddy, Consulting Engineers, August, 1921; also, in paper by John W. Raymond, *Journal*, Boston Soc. of Civ. Engrs., January, 1923, Vol. X, p. 19.

placed five Fergusson recording rain gauges in that city in order to obtain information on this as well as other allied problems. The lack of sufficient gauges, as well as operating difficulties, made it impossible to obtain records of value on the distribution of intense rains.

About 1900, four Fergusson recording rain gauges were placed in Worcester, Mass., under the direction of Mr. Harrison P. Eddy, but no satisfactory data on the distribution of intense rainfall were obtained. The records of the gauges showed wide variations in intensities, for the same storms.

In New York, N. Y., there are eight recording rain gauges, but as they have been placed on an area about 29 miles long and 1½ miles wide, extending from the southerly end of the Borough of Richmond to 177th Street, in the Borough of the Bronx, little opportunity is presented for obtaining records of the type discussed by the writer. This subject has been referred to in a recent report,* as follows:

"A comparison of high intensities at different stations during the same storm indicates how restricted the areas are that are usually subject to the extreme intensities upon which our intensity curves and estimates of run-off are based."

Asa E. Phillips, M. Am. Soc. C. E., in presenting† data obtained from the records of three recording rain gauges spaced at intervals of 2 100 and 1 975 yd., respectively, approximately in a line paralleling the general valley of the Potomac River, makes the following comments:

"This is not, perhaps, the first attempt to compare the records of a series of rain gauges covering a considerable area, and the results are altogether incomplete and tentative, yet this phase of the subject appears to merit discussion, and more extended observations may lead to a better understanding of several of the factors of run-off.

"The feature of these comparative records, the writer would suggest, is not illustrated clearly by the curves of mean rate shown on the diagrams; for, in the latter, the variation in station rate is an important factor; but, rather, the very limited area of maximum precipitation, which is well indicated by the travel of the maximum across the field of observation.

"Whether the great storms of 1898 at Philadelphia and Norfolk which were of extraordinary duration, were recorded correctly by the single station of observation, or whether the average rate for any period, over a considerable area, fell far below that record, it is not possible to determine. Local observations, however, seem to indicate that no storm has here been recorded having an average precipitation for the three stations exceeding 2 in. of actual rainfall in one hour, and no storm of great duration—and therefore considerable area, as city areas are measured—having an average which approached 2 in. in any one hour."

Seven recording gauges have been installed within the city limits of St. Louis, Mo., by W. W. Horner, M. Am. Soc. C. E., and in a paper entitled "Extraordinary Rainfall Flooded Portions of St. Louis",‡ a comparison of the chart records from seven gauges for the storm of August 24, 1918, is shown. These gauges are in a long narrow area, about 6¾ by 1¾ miles, extending in a

* Progress Report of the Committee on Rainfall and Run-off, Kenneth Allen, M. Am. Soc. C. E., Chairman, *The Municipal Engineers Journal*, Vol. 8 (1922), p. 26.

† *Transactions*, Am. Soc. C. E., Vol. LIV (1905), pp. 191–192.

‡ *Engineering News-Record*, October 10, 1918, p. 672.

westerly direction from the Mississippi River and across the city and are spaced from ¾ mile to 1¾ miles apart. The maximum precipitation for 1 hour (11:20 A. M. to 12:20 P. M.) is shown in Table 1 to vary from 3.60 in. to 0.18 in. Unfortunately, records of gauges to the north or south were not given, thus preventing a determination of the area covered by the intense downpour for a given period. The following quotation from Mr. Horner's paper is of interest:

"From the best evidence it would be judged that the intense downpour, at any one time, probably covered an area between 2 000 and 4 000 acres. In addition to the effect of the extraordinary intensity, the effect on the sewer system was accentuated by the movement of the storm in the direction of the flow through the sewer mains and at an approximately equal velocity. It may have been entirely possible, therefore, in so far as the effect on certain main sewers was concerned, for the severe intensity to have been active simultaneously over the whole area."

TABLE 1.—RAINFALL OF AUGUST 24, 1918, at ST. LOUIS, Mo.*

Straight-line distances, in miles.	Rain-gauge station.	Total precipitation in hour of maximum intensity, 11 A. M. to 12:20 P. M., in inches.	Time of beginning of heavy downpour, A. M.
	U. S. Weather Bureau	1.46	Extreme eastern gauge 11:30
6⅝	613 North Garrison Street	3.60	" " " 11:20
	St. Louis University	2.64	" " " 11:10
	Kingsley and Enright	0.55	" " " 10:57
	Union and Patton	0.39	" " " .....
	Shawmut and Ridge	0.25	" " " .....
	University City	0.18	Extreme western gauge 10:55

* Data from *Engineering News-Record*, October 10, 1918, p. 672.

Rainfall records have been obtained at New Orleans from six recording gauges operated since 1907, by the Sewerage and Water Board under the direction of George G. Earl, M. Am. Soc. C. E. These gauges cover an area about 5 by 3½ miles and are spaced from 1½ to 3 miles apart. The records from four typical storms of different characters have been selected and made available for this study with the assistance of Alfred F. Theard, M. Am. Soc. C. E., and are referred to subsequently. Some interesting comparative studies based on the records of these gauges have been published.*

Messrs. Metcalf and Eddy refer† to the distribution of intense rainfall, as follows:

"*Coefficient of Distribution of Rainfall.*—It is a well-recognized fact that heavy rains cover but a limited area, and the intensity of downpour diminishes as the distance from the center of the storm increases. Very little definite information is to be had regarding these matters, and that little has not been analyzed sufficiently to draw positive conclusions, other than that there is such a diminution in rate of rainfall. American engineers have usually been content to recognize the fact, and to allow for it by using a smaller run-off factor for larger areas."

* "The Distribution of Rainfall Over Restricted Areas", by Alfred J. Henry, *Monthly Weather Review*, July, 1921, p. 401.

† "American Sewerage Practice", Vol. I, p. 269.

Frühling states* that, according to observations in Breslau, Germany, the rate of precipitation at a distance of 3 000 m. (9 850 ft.) from the center of the storm was one-half the maximum rate, and that a plotting of the reduction in intensity followed a parabolic curve. From these data, he derived the equation, $D = 1—0.005 \sqrt{L}$ (for L, in meters), assuming the center of the storm to be in the center of the drainage area. D represents the ratio of the average intensity of precipitation over an area of diameter, L, to the maximum intensity at the center. L is the length of the sewer, or diameter of the drainage area. If L is expressed in feet, this equation reduces to, $D = 1—0.0028 \sqrt{L}$.

Frühling does not give the data on which he based his assumptions, but apparently his studies were confined to intensities in excess of 55 sec-liters per hectare (0.78 in. per hour). From the foregoing equation, the following values of D have been computed for various values of L, in feet:

$L =$	1 000	2 500	10 000	15 000	20 000
$D =$	0.91	0.86	0.72	0.66	0.60

Daniel W. Mead, M. Am. Soc. C. E., refers to a 90-min. precipitation which occurred near Cambridge, Ohio, on July 16, 1914.† The storm centered over the U. S. Weather Bureau Station and a good record of the maximum intensity (7.09 in. in 90 min., equivalent to a rate of 4.73 in. per hour) was obtained. Shortly after the storm, the County Surveyor traced its outline, thus giving a reasonably good measure of the area covered. It was assumed that the outline represented a rainfall of ½ in. Isohyetal lines were plotted proportionately between this outline and the Weather Bureau Station. The areas shown in Table 9 were measured on the diagram in the book referred to previously.

The apparent need for definite information in regard to the area on which high intensities of precipitation may be expected for short periods of time caused Messrs. Metcalf and Eddy to take up the work of collecting the available data in the Boston Metropolitan District. Several recording rain gauges have been established recently in this District, which, together with gauges already placed, have made possible the collection of valuable data.

Recording Rain Gauges in the Boston Metropolitan District

The recording rain gauges from which records have been obtained are listed in Table 2, together with three others which have not been particularly helpful. At the time of these studies, the Newton gauge had not been placed in operation; the one at Lynn is more than 6 miles from the nearest gauge and, therefore, the records are not particularly helpful in this study. As the Blue Hill gauges are at such an elevation above sea level (Elevation 635) and also about 5 miles from the nearest gauge, these records will not be of much value until an intermediate gauge is established.

* Handbuch der Ingenieurwissenschaften, III Wasserbau, IV Band, 4 Auflage, 22–24.

† "Hydrology", pp. 245–246; see, also, "The Ohio Water Problems", by C. E. Sherman, *Bulletin No. 15*, Coll. of Eng., Univ. of Ohio, Columbus, Ohio.

TABLE 2.—RECORDING RAIN GAUGES IN THE BOSTON METROPOLITAN DISTRICT.

Place and location of gauge.	Type of gauge.	Approximate elevation of gauge above mean sea level, in feet.	Approximate distance above ground, in feet.	Diameter of collecting rim, in inches.	Maintained by:
Brookline: Town Hall (roof).	Friez tipping bucket	144	93	12	Town Engineer, Brookline
City proper: Post Office Building (roof)	Marvin (weighing)	170	154	8	U. S. Weather Bureau
City proper: Albany Street (roof)	Fergusson (old style)	25	15.5	8	Boston Sewer Service
East Boston: East Eagle Street (roof of stable)	Fergusson (old style)	44	24.5	8	Boston Sewer Service
Dorchester: Gibson Street (roof of stable)	Fergusson (old style)	37	24.5	8	Boston Sewer Service
Jamaica Plain: Child Street (roof of shed)	Fergusson (old style)	60	14.5	8	Boston Sewer Service
Moon Island: Boston Harbor (roof)	Fergusson (old style)	33	10	8	Boston Sewer Service
Brighton: Chestnut Hill (shop roof, Metropolitan Water-Works Pumping Station)	FitzGerald	...	.....	14.85	Metropolitan District Commission, Water Division
Cambridge: City Hall (Guaranty Trust Company Building roof)	Fergusson (old style)	73	52	8	City Engineer
West Roxbury: Boston (Platform on ground, Brookline Water-Works Pumping Station)	Friez tipping bucket	119	9	12	Town Engineer, Brookline
Lynn: City Hall (roof)*	Friez tipping bucket	103	72.5	12	City Engineer
Milton: Blue Hill Meteorological Observatory*	Draper and Fergusson	635	1	8	Blue Hill Meteorological Observatory
Newton: City Hall†	Fergusson (old style)	Not in place.		8	City Engineer

* Records not particularly helpful until intermediate gauges have been established.
† No records obtained.

With the exception of the Blue Hill gauge, all the gauges are within 10 miles of the State House.

In order to derive information of value on the distribution of rainfall intensity from these gauge records, it is essential that the center of intensity of a storm must pass, not only through the area served by these gauges, but, also, near the center of the area so that measures of the decrease in intensity may be available on all sides of the intense peak for any given period of time. With the gauges as situated, it is necessary for the center of intensity to pass near the Brookline Town Hall or the Albany Street gauges. Prior to 1918, when the Brookline Town Hall gauge was installed, there was not a sufficient number of gauges in operation to produce records of value for these studies.

RECORDS OF STORMS FROM WHICH INFORMATION HAS BEEN OBTAINED

The records of all storms from 1918 to 1922, inclusive, in which intensities equal to or greater than 0.70 in. per hour for a period of 15 min. were recorded, have been examined. Those storms have been selected for which records are available from all or nearly all the gauges, in order to serve as a

basis for a study of the distribution of intense rainfall. The number of such storms for which trustworthy records were found is rather small, because in many cases one or more important gauges were out of commission just when they were most needed. However, the records of twenty storms have been analyzed and an abbreviated tabulation has been prepared (Table 3), showing the amount and rate of precipitation during certain significant periods of each storm. Where the time on the charts was known to be in error, it has been corrected so as to make the records, as nearly as possible, synchronous in this respect.

Storm of July 9, 1921.—The storm of July 9, 1921, from which all the more valuable records were obtained, produced unusual rainfall intensities. The storm, which was of the thunder-storm type, caused extensive damage from flooding in Boston and vicinity. During this storm, both the Post Office gauge and the Cambridge gauge recorded the greatest amounts for 24 hours on record at these stations, as follows: Post Office gauge (U. S. Weather Bureau), 6.04 in., and the Cambridge City Hall gauge, 6.75 in.

Fortunately, for the purposes of this study, the recording rain gauges caught some of the downpours which were so located as to be of value. If the area served by gauges had been greater, perhaps other intensities would have been recorded greater than those given subsequently.

Accuracy of Records.—Most of the recording rain-gauge collectors are on the roofs of buildings. The Post Office gauge (U. S. Weather Bureau) and the Cambridge gauge are on buildings having flat roofs and parapet walls. The Boston Sewer Service gauges are not so well protected from wind, and the collectors are connected to the weighing mechanism by 1½-in. rubber pressure hose, or 1½-in. galvanized iron pipe, from 4 to 10 ft. in length. Although this construction is open to criticism, it should be noted that in every case in which records have been used, the downpour occurred a sufficient time after the beginning of the storm to ensure the thorough wetting of the inside of the hose or pipe, thus reducing the time required for the transmission of the rain from the collector to the weighing can. The Brookline Town Hall gauge is on the roof and doubtless is affected to some extent because of lack of wind protection. Other features of the exposure of these gauges may be criticized to some extent, but the conditions are probably as good as can be expected.

The records are also subject to errors due to the personal equation of the attendants. A larger number of records would have been obtained if all the gauges could have received the careful and intelligent supervision given some of them. Charts were discarded if there was doubt about the accuracy of the rainfall recorded, or the time of the chart, or any other important feature. In this manner, the records of a number of important storms were lost.

A comparison of the total rainfall recorded by each of the gauges for a number of storms, however, showed no evidence tending to discredit the records of any one gauge. Although it is recognized that the records used are subject to errors of various kinds, it is believed that these errors are not sufficient to effect seriously the results or to impair the value of tentative conclusions which may be drawn from them.

TABLE 3.—PRECIPITATION DURING SELECTED PERIODS FOR CERTAIN INTENSE STORMS IN THE BOSTON METROPOLITAN DISTRICT (1918-1922), AS RECORDED BY AUTOMATIC RAIN GAUGES.

(Location of Gauges, Precipitation, in Inches, and Rate of Precipitation, in Inches per Hour, for Periods Stated.)

Date.	Time (Eastern Standard) From	Time (Eastern Standard) To	Period of time, in minutes.	Storm reference number.	Brookline Town Hall.		City Proper, U. S. Weather Bureau Post Office Building.		City Proper, Albany Street.		East Boston, East Eagle Street.		Dorchester, Gibson Street.		Jamaica Plain, Child Street.		Boston Harbor, Moon Island.		Brighton, Chestnut Hill Metropolitan Water-Works.		Cambridge, City Hall.		West Roxbury, Brookline Water-Works Pumping Station.		Lynn, City Hall.			Milton, Blue Hill.	
					Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.		Inches.	Inches per hour.	
Sept. 13, 1918	4:05 A.M.	4:20 A.M.	15	1	0.28	1.12	0.15	0.60	0.15	0.60	0.04	0.16	0.32	1.28	0.01	0.04	0.20	0.80	0.16	0.64	0.08	0.32	....	....	....	....	....	....	....
July 22, 1919	1:25 P.M.	1:40 P.M.	15	2	0.39	1.56	0.25	1.00	0.20	0.80	0	0	0.12	0.48	0.02	0.08	0	0	0.03	0.12	0	0	....	....	....	....	....	....	....
"	1:15 P.M.	1:45 P.M.	30	3	0.43	0.86	0.35	0.70	0.41	0.82	0	0	0.13	0.26	0.20	0.40	0	0	0.03	0.06	0	0	....	....	....	....	....	....	....
Nov. 23, 1920	10:05 A.M.	10:20 A.M.	15	4	0.28	1.12	0.04	0.16	0.06	0.24	....	....	0.15	0.60	0.09	0.36	....	....	0.12	0.48	0.08	0.32	....	....	....	....	....	....	....
"	10:00 A.M.	10:30 A.M.	30	5	0.40	0.80	0.18	0.36	0.16	0.32	....	....	0.28	0.56	0.18	0.36	....	....	0.29	0.58	0.30	0.60	....	....	....	....	....	....	....
July 9, 1921	2:10 A.M.	2:25 A.M.	15	6	0.64	2.56	0.46	1.84	0.43	1.72	....	....	0.03	0.12	....	....	....	....	0.58	2.32	0.38	1.52	....	....	0.01	0.04	0.09	0.36	
"	2:35 A.M.	2:50 A.M.	15	7	0.62	2.48	0.42	1.68	0.18	0.72	....	....	0.40	1.60	....	....	....	....	0.30	1.20	0.07	0.28	....	....	0.06	0.24	0.10	0.40	
"	8:05 A.M.	8:20 A.M.	15	8	0.80	3.20	0.33	1.32	0.37	1.48	0.27	1.08	0.42	1.68	0.06	0.24	....	....	0.01	0.04	0.07	0.28	....	....	0.05	0.20	0.02	0.08	
"	2:15 A.M.	2:45 A.M.	30	9	1.20	2.40	0.50	1.00	0.51	1.02	....	....	0.18	0.36	....	....	....	....	0.98	1.96	0.36	0.72	....	....	0.10	0.20	0.22	0.44	
"	7:50 P.M.	8:20 P.M.	30	10	1.19	2.38	0.57	1.14	0.68	1.36	0.95	1.90	0.43	0.86	0.66	1.32	....	....	0.66	1.32	0.27	0.54	....	....	0.13	0.26	0.09	0.18	
"	2:10 A.M.	2:55 A.M.	45	11	1.63	2.17	1.08	1.44	0.78	1.04	....	....	0.63	0.84	....	....	....	....	1.16	1.55	0.58	0.77	....	....	0.11	0.15	0.26	0.35	
"	7:45 P.M.	8:30 P.M.	45	12	1.29	1.72	0.72	0.96	0.92	1.23	1.16	1.55	0.51	0.68	1.04	1.39	....	....	0.78	1.04	0.35	0.47	....	....	0.19	0.25	0.10	0.13	
"	2:05 A.M.	3:05 A.M.	60	13	1.72	1.72	1.37	1.37	0.95	0.95	....	....	1.02	1.02	....	....	....	....	1.37	1.37	0.72	0.72	....	....	0.11	0.11	0.32	0.32	
"	7:35 P.M.	8:35 P.M.	60	14	1.29	1.29	0.81	0.81	0.97	0.97	1.19	1.19	0.58	0.58	1.14	1.14	....	....	0.97	0.97	0.56	0.56	....	....	0.40	0.40	0.22	0.22	
Mar. 7, 1922	8:45 P.M.	9:00 P.M.	15	15	0.19	0.76	0.01	0.04	0.14	0.56	0.02	0.08	0	0	0	0	0.04	0.16	0.02	0.08	0.09	0.36	0.09	0.36	....	....	....	....	
June 19, 1922	3:15 A.M.	3:30 A.M.	15	16	0.23	0.92	0.03	0.12	0.02	0.08	0.02	0.08	....	....	0.07	0.28	0.08	0.32	0.13	0.52	0.03	0.12	0.13	0.52	....	....	....	....	
"	3:20 A.M.	3:50 A.M.	30	17	0.36	0.72	0.12	0.24	0.05	0.10	0.13	0.26	....	....	0.18	0.36	0.16	0.32	0.16	0.32	0.27	0.54	0.25	0.50	....	....	....	....	
Aug. 4, 1922	3:45 P.M.	4:00 P.M.	15	18	0.54	2.16	0	0	0	0	0	0	0	0	0	0	0	0	0.20	0.80	0.02	0.08	....	....	....	....	....	....	
"	3:45 P.M.	4:15 P.M.	30	19	0.85	1.70	0.10	0.20	0.11	0.22	0	0	....	....	0.04	0.08	0	0	0.40	0.80	0.11	0.22	....	....	....	....	....	....	
Sept. 16, 1922	12:45 A.M.	1:00 A.M.	15	20	0.32	1.28	0.19	0.76	0.39	1.56	0.07	0.28	0.11	0.44	0.08	0.32	0	0	0.02	0.08	0.22	0.88	....	....	....	....	....	....	

TABLE 4.—AVERAGE RATES OF PRECIPITATION AND AREAS COVERED.

Date.	Time (Eastern Standard): From:	To:	Period of time, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate / Maximum rate.	Storm reference number.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
September 13, 1918..	5:05 A.M.	5:20 A.M.	15	1.11	6	0.01	1.12	0.99	1
				1.05	275	0.43	"	0.94	
				0.98	1 030	1.61	"	0.87	
				0.91	2 295	3.59	"	0.81	
				0.84	4 006	6.26	"	0.75	
July 22, 1919.....	1:25 P.M.	1:40 P.M.	15	1.53	40	0.06	1.56	0.98	2
				1.46	313	0.49	"	0.94	
				1.39	820	1.28	"	0.89	
				1.32	1 545	2.41	"	0.85	
				1.26	2 460	3.84	"	0.81	
				1.19	3 525	5.51	"	0.76	
				1.13	4 875	7.62	"	0.72	
				1.06	6 330	9.90	"	0.68	
				1.00	7 880	12.31	"	0.64	
				0.94	9 502	14.84	"	0.60	
				0.88	11 220	17.53	"	0.56	
	1:15 P.M.	1:45 P.M.	30	0.83	1 276	1.99	0.96	0.97	3
				0.78	3 240	5.07	"	0.91	
				0.73	5 515	8.61	"	0.85	
				0.67	8 200	12.83	"	0.78	
				0.61	11 300	17.66	"	0.71	
				0.55	14 950	23.38	"	0.64	
November 23, 1920.	10:05 A.M.	10:20 A.M.	15	1.11	22	0.03	1.12	0.99	4
				1.05	289	0.45	"	0.94	
				0.98	902	1.41	"	0.88	
				0.91	1 880	2.94	"	0.82	
				0.85	3 172	4.96	"	0.76	
				0.78	4 935	7.71	"	0.69	
				0.71	7 200	11.25	"	0.63	
	10:00 A.M.	10:30 A.M.	30	0.75	1 266	1.98	0.80	0.94	5
				0.68	4 880	7.64	"	0.84	
July 9, 1921.....	2:10 A.M.	2:25 A.M.	15	2.53	120	0.19	2.56	0.99	6
				2.47	693	1.08	"	0.96	
				2.39	1 840	2.88	"	0.93	
	2:35 A.M.	2:50 A.M.	15	2.44	73	0.11	2.48	0.98	7
				2.37	347	0.54	"	0.96	
				2.30	825	1.29	"	0.93	
				2.23	1 485	2.32	"	0.90	
				2.16	2 495	3.90	"	0.87	
				2.09	3 655	5.71	"	0.84	
				2.02	5 136	8.03	"	0.81	
				1.96	6 665	10.41	"	0.79	
	8:05 P.M.	8:20 P.M.	15	3.15	22	0.03	3.20	0.99	8
				3.06	93	0.15	"	0.96	
				3.00	209	0.33	"	0.94	
				2.93	378	0.59	"	0.92	
				2.87	582	0.91	"	0.90	
				2.80	858	1.34	"	0.88	
				2.74	1 138	1.78	"	0.86	
				2.67	1 475	2.31	"	0.84	
				2.59	1 902	2.97	"	0.81	
				2.53	2 348	3.67	"	0.79	
				2.47	2 795	4.37	"	0.77	
				2.41	3 293	5.15	"	0.75	
				2.33	3 935	6.15	"	0.73	
				2.26	4 616	7.21	"	0.71	
				2.19	5 300	8.28	"	0.69	
				2.13	5 970	9.33	"	0.67	
				2.07	6 700	10.47	"	0.65	

TABLE 4.—(*Continued*).

Date.	Time (Eastern Standard): From:	To:	Period of time, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate / Maximum rate.	Storm reference number.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
July 9, 1921	2:15 A.M.	2:45 A.M.	30	2.35	156	0.24	2.40	0.98	9
				2.27	582	0.91	"	0.95	
				2.20	1 267	1.98	"	0.92	
				2.13	2 292	3.59	"	0.89	
	7:50 P.M.	8:20 P.M.	30	2.34	44	0.07	2.38	0.98	10
				2.26	231	0.36	"	0.95	
				2.20	538	0.84	"	0.92	
				2.13	1 013	1.59	"	0.89	
				2.06	1 615	2.52	"	0.87	
				1.99	2 379	3.72	"	0.84	
				1.93	3 222	5.04	"	0.81	
				1.86	4 300	6.73	"	0.78	
				1.79	5 535	8.65	"	0.75	
				1.72	6 915	10.80	"	0.72	
	2:10 A.M.	2:55 A.M.	45	2.14	89	0.14	2.17	0.99	11
				2.07	453	0.71	"	0.95	
				1.99	1 174	1.83	"	0.92	
				1.93	2 160	3.38	"	0.89	
				1.86	3 548	5.54	"	0.86	
				1.79	5 165	8.07	"	0.83	
				1.72	7 280	11.37	"	0.79	
	7:45 P.M.	8:30 P.M.	45	1.70	27	0.04	1.72	0.99	12
				1.65	367	0.57	"	0.96	
				1.58	1 173	1.83	"	0.92	
				1.52	2 470	3.86	"	0.88	
				1.44	4 230	6.61	"	0.84	
				1.38	6 180	9.66	"	0.80	
	2:05 A.M.	3:05 A.M.	60	1.71	22	0.03	1.72	0.99	13
				1.65	614	0.96	"	0.96	
				1.58	2 022	3.16	"	0.92	
				1.51	4 298	6.71	"	0.88	
				1.44	7 200	11.55	"	0.84	
	7:35 P.M.	8:35 P.M.	60	1.25	643	1.00	1.29	0.97	14
				1.17	2 525	3.95	"	0.91	
				1.11	5 530	8.64	"	0.86	
				1.04	9 800	15.30	"	0.81	
March 7, 1922	8:45 P.M.	9:00 P.M.	15	0.73	182	0.28	0.76	0.96	15
				0.66	1 455	2.27	"	0.87	
				0.60	3 180	4.97	"	0.79	
				0.54	5 650	8.84	"	0.70	
				0.47	9 050	14.15	"	0.61	
June 19, 1922	3:15 A.M.	3:30 A.M.	15	0.91	27	0.04	0.92	0.99	16
				0.85	577	0.90	"	0.93	
				0.78	1 869	2.91	"	0.85	
				0.71	3 970	6.20	"	0.77	
				0.65	6 730	10.50	"	0.70	
	3:20 A.M.	3:50 A.M.	30	0.71	44	0.07	0.72	0.99	17
				0.65	1 960	3.06	"	0.90	
				0.58	5 930	9.25	"	0.81	
August 4, 1922	3:45 P.M.	4:00 P.M.	15	2.13	13	0.02	2.16	0.99	18
				2.06	62	0.10	"	0.96	
				2.01	133	0.21	"	0.93	
				1.93	267	0.42	"	0.89	
				1.85	458	0.72	"	0.86	
				1.78	698	1.09	"	0.83	
				1.71	1 036	1.62	"	0.79	
				1.65	1 338	2.09	"	0.76	

TABLE 4.—(*Continued*).

Date.	Time (Eastern Standard): From:	To:	Period of time, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate / Maximum rate.	Storm reference number.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
August 4, 1922.....	3:45 P.M.	4:00 P.M.	15	1.59	1 699	2.65	2.16	0.73	18
				1.52	2 125	3.32	"	0.70	
				1.45	2 615	4.09	"	0.67	
				1.38	3 188	4.98	"	0.64	
				1.31	3 798	5.98	"	0.61	
				1.24	4 440	6.93	"	0.58	
	3:45 P.M.	4:15 P.M.	30	1.65	40	0.06	1.70	0.97	19
				1.57	196	0.31	"	0.93	
				1.50	454	0.71	"	0.89	
				1.44	782	1.22	"	0.85	
				1.37	1 222	1.91	"	0.81	
				1.30	1 788	2.79	"	0.77	
				1.23	2 480	3.88	"	0.72	
				1.17	3 232	5.05	"	0.69	
				1.10	4 040	6.31	"	0.65	
September 16, 1922..	12:45 A.M.	1:00 A.M.	15	1.53	98	0.15	1.56	0.98	20
				1.46	630	0.99	"	0.94	
				1.39	1 705	2.66	"	0.89	
				1.33	2 942	4.60	"	0.85	
				1.27	4 275	6.68	"	0.82	
				1.21	5 880	9.20	"	0.78	
				1.15	7 690	12.02	"	0.74	

Method of Analysis.—In studies of rainfall and run-off for storm-water drain design, the period of duration of rain, or the period of concentration of run-off, is seldom less than 15 min., or more than 60 min.

Analyses of these rainfall records therefore, have been limited to periods of 15, 30, 45, and 60 min. Even for 60 min., only two cases were found in which the distribution of the intense rates could be satisfactorily studied, because the area covered by the downpour was larger or different from that served by the gauges.

In analyzing the records for each storm, a table was prepared showing the precipitation which accumulated in a number of successive 15, 30, 45, and 60-min. periods, each gauge period being synchronous with the other comparable gauge periods. From these, a selection was made of those periods showing the highest intensity at the Brookline Town Hall or Albany Street gauges. These periods are shown in Table 3, together with the corresponding accumulated precipitation and rate of precipitation for each station.

The approximate distribution of the intensities was obtained by plotting the rates of precipitation on a base plan, showing the geographical location of the several rain-gauge stations. Lines of equal rainfall intensity, or isodyhyetals, were then plotted proportionately between the stations in much the same manner that topographical contours are plotted from stadia survey elevations. The resulting drawings considerably reduced are shown in Figs. 1 to 13, inclusive, Plate I.

Downpours for which closed isodyhyetals could not be drawn, because of insufficient data, were discarded. The area enclosed by each line was determined by planimeter, and the area and average depth of precipitation over that area were computed. (See Table 4, Column 5). The average depth of precipitation over each area was computed by the method used in the rainfall studies of the Miami Conservancy District.*

The ratio of the average rate of precipitation over a given area to the maximum rate is given in Table 4, Column 9. Although there appear to be definite relations between the average rate of precipitation and the maximum for given areas, other factors such as the frequency of occurrence of the downpour and its duration complicate these relations and make it impossible to show a comparison by a single curve or formula such as that of Frühling, referred to previously. Furthermore, the method of comparison given subsequently seems to offer a more practical method of using the data. The ratios are given in Table 4, in order to aid in further studies of this kind.

Comparisons of the average rates of precipitation (Table 4, Column 5,) for periods of 15, 30, 45, and 60-min. duration, respectively, with the areas covered, are shown in Figs. 1, 2, 3, and 4, inclusive. The approximate frequency of the several downpours was estimated by comparing the maximum intensities recorded during each period, with the estimated frequency of occurrence of similar rates as shown by the 38-year records of the Chestnut Hill recording rain gauge, prepared by Mr. Harrison P. Eddy, and herein inserted as Table 5.† The estimated frequency thus obtained is of assistance in interpreting the variations in area covered by downpours of the same intensity and duration.

TABLE 5.—Co-Ordinates of Precipitation Curves of Various Frequencies, According to the Chestnut Hill Records, Thirty-eight Years, 1879-1916, Inclusive.

(Intensity, in Inches.)

Duration, in minutes.	Frequency.						
	Maximum, 38 years.	20 years.	15 years.	10 years.	5 years.	3 years.	2 years.
5	9.10	7.50	7.40	5.80	4.56	3.97	3.71
10	6.70	5.17	4.85	4.57	3.52	3.03	2.81
15	5.40	4.17	3.82	3.53	2.84	2.52	2.27
20	4.65	3.59	3.31	3.00	2.42	2.17	1.97
30	3.60	2.75	2.65	2.48	1.87	1.75	1.53
45	2.58	2.08	2.00	1.94	1.58	1.37	1.24
60	2.06	1.79	1.65	1.60	1.35	1.15	1.05
80	1.70	1.48	1.35	1.30	1.11	0.92	0.85
100	1.45	1.29	1.18	1.06	0.89	0.76	0.70
120	1.29	1.20	1.02	0.88	0.75	0.66	0.61
150	1.13	1.10	0.87	0.72	0.62	0.57	0.51
180	1.01	1.01	0.79	0.65	0.54	0.50	0.45

* "Storm Rainfall of Eastern United States", Technical Reports, Part V, p. 225, The Miami Conservancy District, Ohio.

† *Journal*, Boston Soc. Civ. Engrs., Vol. 7, No. 2, February, 1920, Table 5, p. 70.

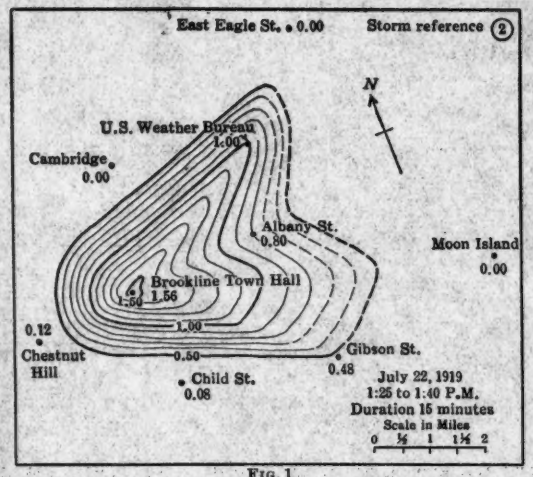

FIG. 1

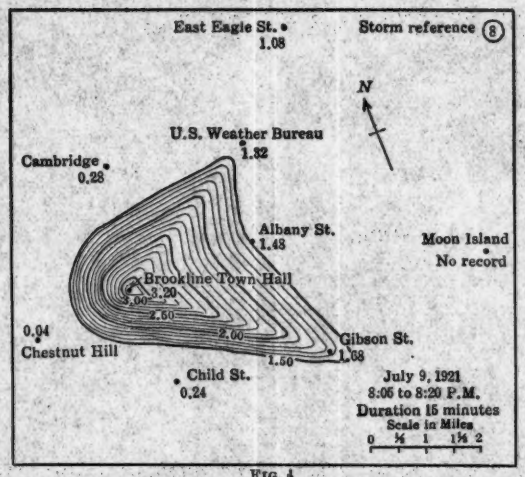

FIG. 4

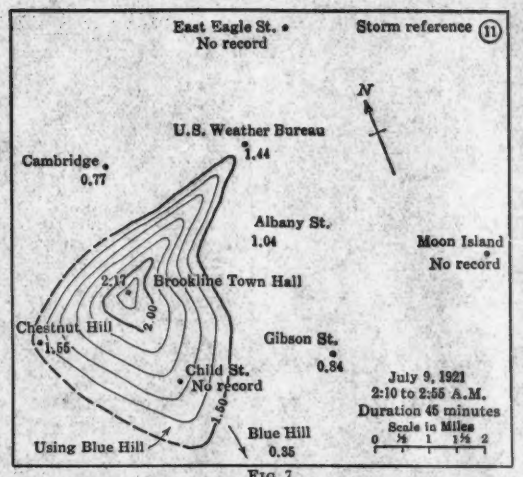

FIG. 7

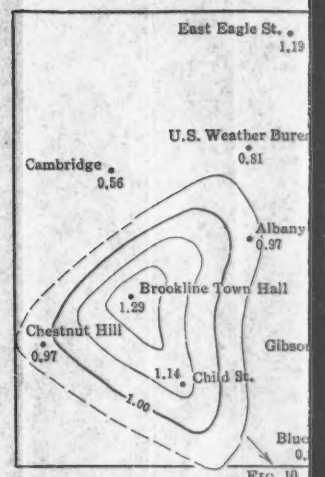

FIG. 10

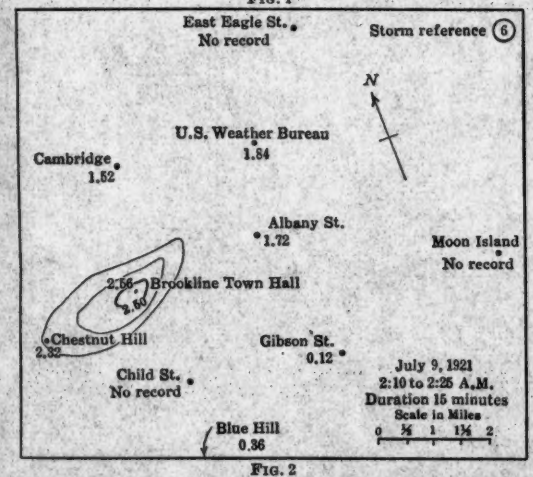

FIG. 2

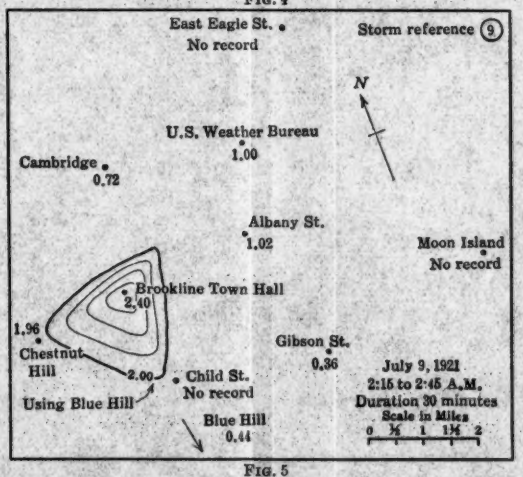

FIG. 5

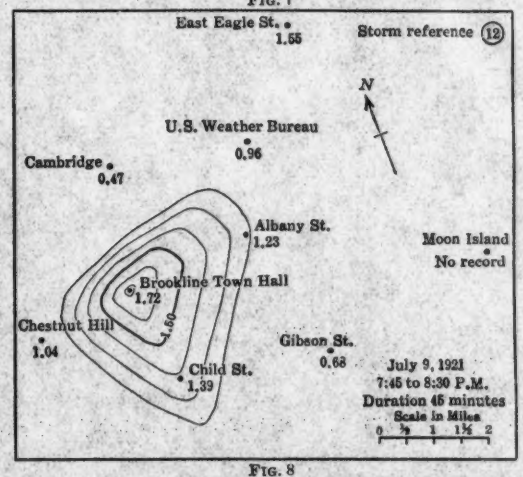

FIG. 8

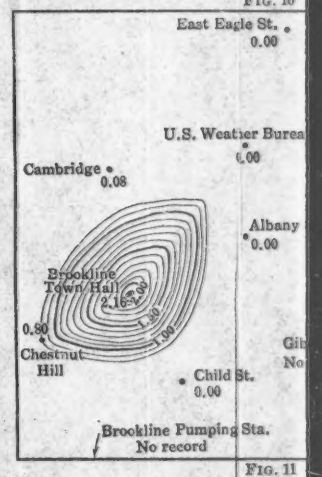

FIG. 11

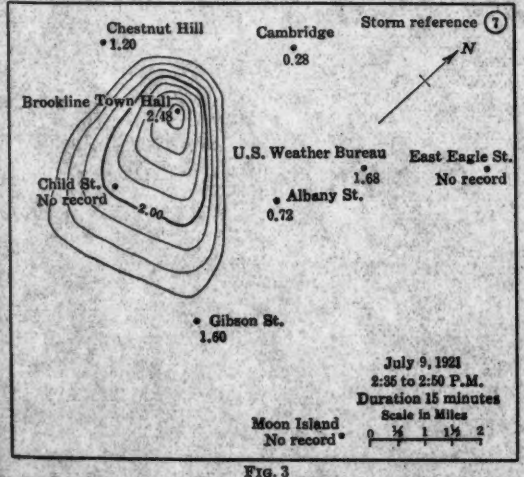

FIG. 3

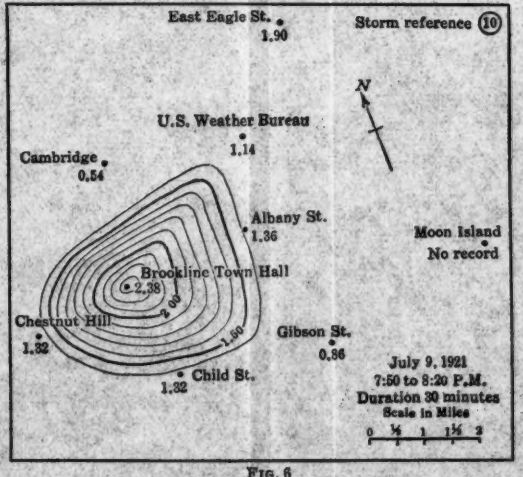

FIG. 6

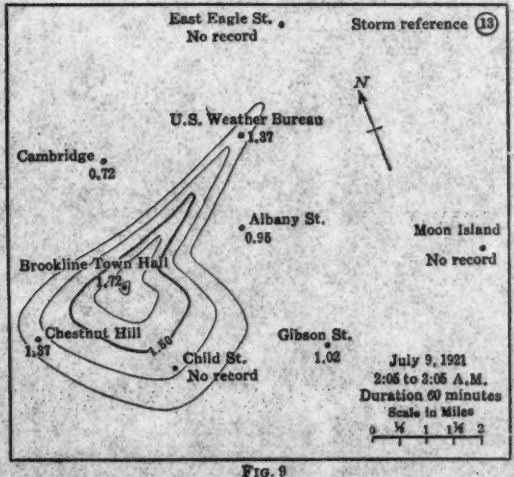

FIG. 9

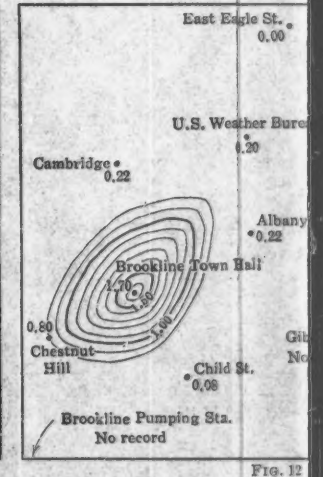

FIG. 12

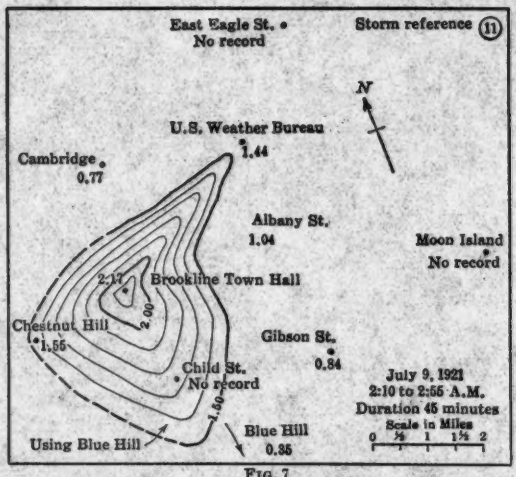

FIG. 7

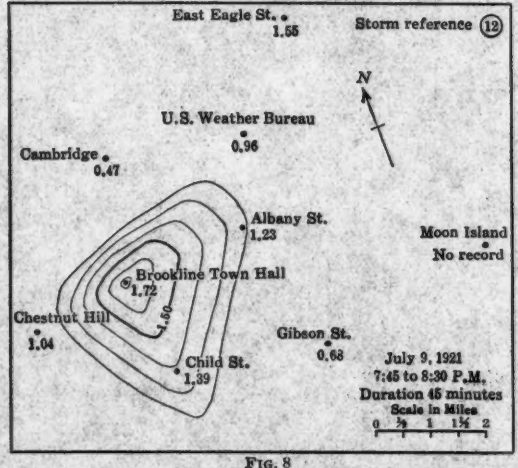

FIG. 8

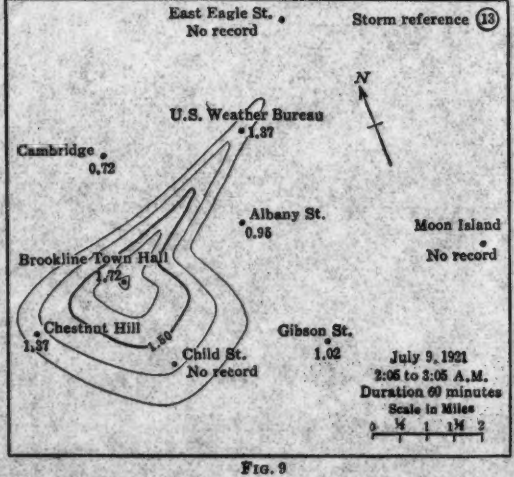

FIG. 9

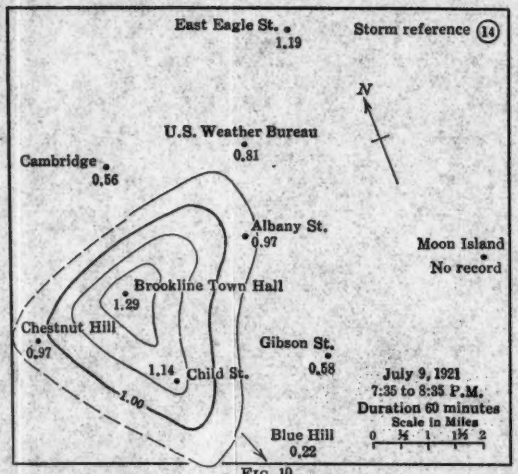

FIG. 10

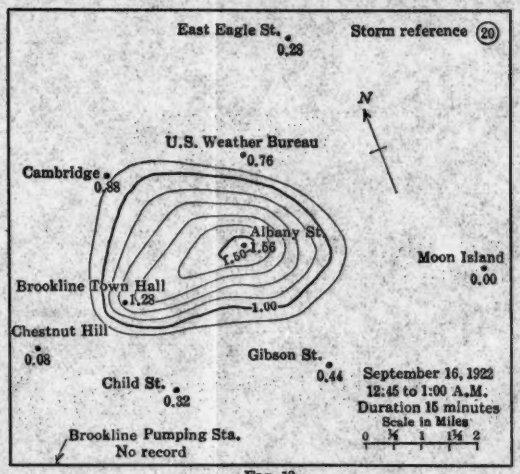

FIG. 13

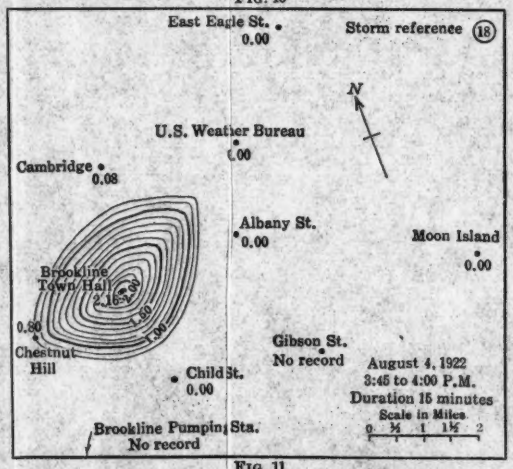

FIG. 11

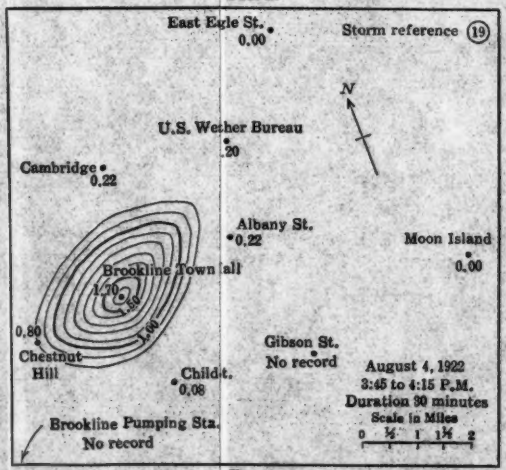

FIG. 12

RAINFALL INTENSITIES IN INCHES PER HOUR AT BOSTON, MASSACHUSETTS

Pap

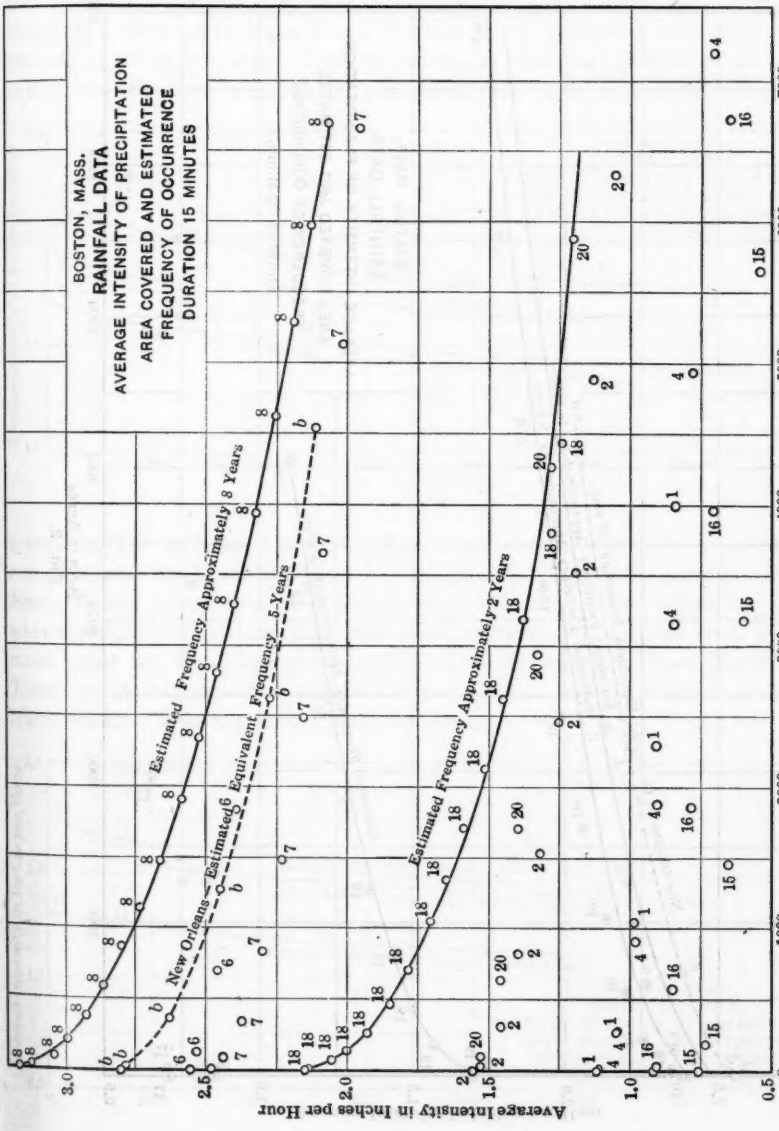
BOSTON, MASS.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AREA COVERED AND ESTIMATED
FREQUENCY OF OCCURRENCE
DURATION 15 MINUTES
Estimated Frequency Approximately 8 Years
New Orleans Estimated Equivalent Frequency 5 Years
Estimated Frequency Approximately 2 Years
Average Intensity in Inches per Hour
Area in Acres
0
1000
2000
3000
4000
5000
6000
7000
0.5
1.0
1.5
2.0
2.5
3.0

FIG. 1.

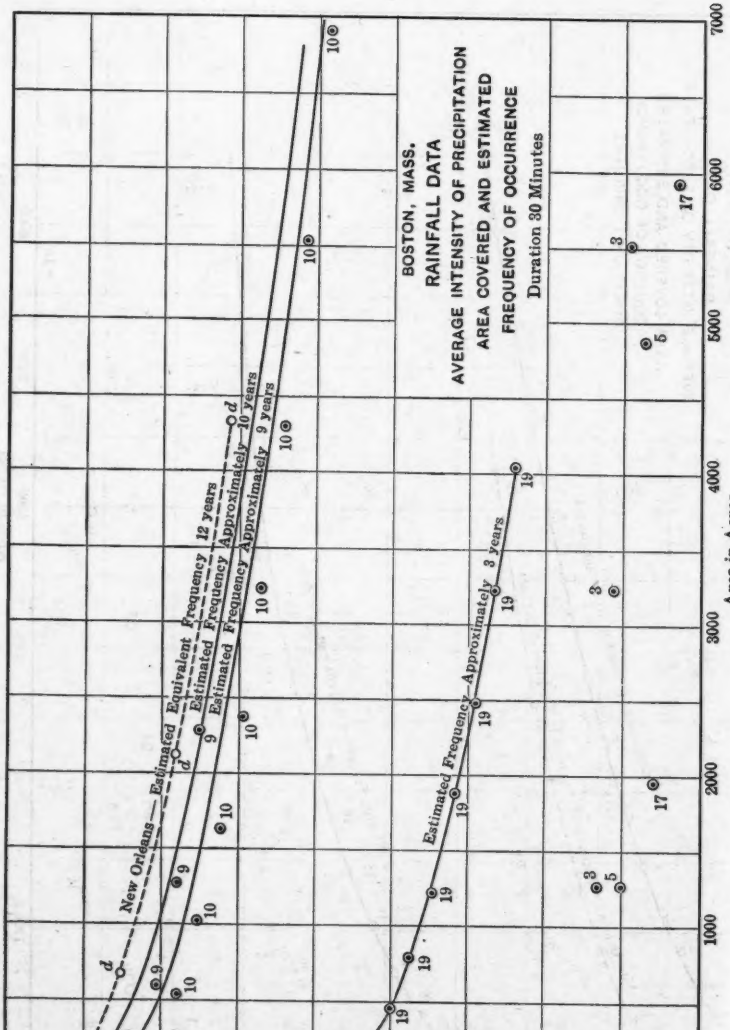
BOSTON, MASS.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AREA COVERED AND ESTIMATED
FREQUENCY OF OCCURRENCE
Duration 30 Minutes
New Orleans — Estimated Equivalent Frequency 12 years
Estimated Frequency Approximately 10 years
Estimated Frequency Approximately 9 years
Estimated Frequency Approximately 3 years
Average Intensity in Inches per Hour
Area in Acres
2.5
2.0
1.5
1.0
0.5
1000
2000
3000
4000
5000
6000
7000

FIG. 2.

The area served by the recording rain gauges in the Boston District is not sufficiently large to furnish information on the location and character of intense downpours which may have occurred in areas adjoining those plotted in the preceding diagrams. That such other downpours occurred at times, is evidenced in Figs. 6 and 8, Plate I, which show isodyhyetals for the periods of 30 and 45 min. (7:50 to 8:20 P. M., and 7:45 to 8:30 P. M., respectively), for the severe storm of July 9, 1921. It should be noted that the

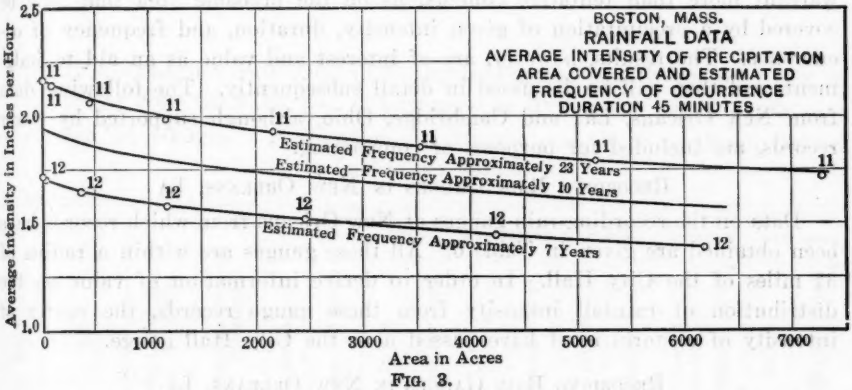

FIG. 3.

second period includes the first. The maximum intensity in each case was recorded at the Brookline Town Hall gauge. For both periods the East Eagle Street gauge recorded higher intensities than the adjoining gauges, which indicates that there may have been an intense downpour with its maximum point for the given periods, located to the north of East Eagle Street. These two downpours were sufficiently far apart to have prevented an accumulative effect. This may not always be the case.

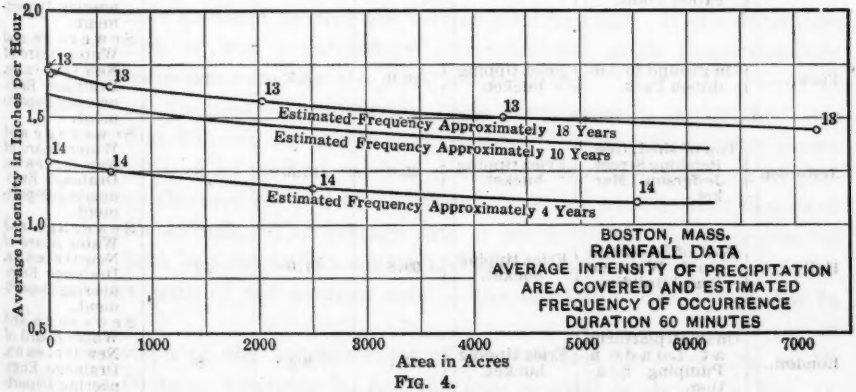

FIG. 4.

The facts presented in this paper indicate that the more intense the storm the greater usually will be the area covered by a given intensity, and the area covered may vary according to the estimated frequency. In this paper, frequency is measured by the period of years during which, as an average, a

certain rate of precipitation for a given duration will occur but once at any one station.

The diagrams presented show that, during excessive downpours, rain does not fall with uniform intensity over any considerable area, and that the average rate of precipitation over drainage areas, of, say, 500 acres and larger, is appreciably less than the maximum rate indicated by one gauge.

The records from which these diagrams have been made are too few to warrant more than tentative conclusions on the probable area that may be covered by a precipitation of given intensity, duration, and frequency of occurrence. The results, however, are of interest and value as an aid to judgment, and they will be discussed in detail subsequently. The following data from New Orleans, La., and Cambridge, Ohio, although supported by fewer records, are included for purposes of comparison.

Recording Rain Gauges in New Orleans, La

Data on the recording rain gauges at New Orleans from which records have been obtained are given in Table 6. All these gauges are within a radius of 3¾ miles of the City Hall. In order to derive information of value on the distribution of rainfall intensity from these gauge records, the center of intensity of a storm must have passed near the City Hall gauge.

Recording Rain Gauges in New Orleans, La.

Street.	Location of gauge.	Type of gauge.	Approximate elevation above mean sea level, in feet.	Approximate distance above ground, in feet.	Diameter of collecting rim, in inches.	Maintained by:
Dublin...	On retaining wall at Water Purification Plant.	Friez tipping bucket.	22.6	21.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Park.....	On ground in Audubon Park.	Friez tipping bucket.	16.0	3.5	12	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Jefferson	Top of shed, Gen. Pershing Street, Jefferson Market.	Friez tipping bucket.	25.0	15.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Hall.....	Roof of Sewerage and Water Board Building.	Friez tipping bucket.	99.8	91.0	12	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
London..	On small platform at London Pumping Station.	Friez tipping bucket.	27.1	8.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.
Algiers..	Top of shed in Algiers, right bank of Mississippi.	Friez tipping bucket.	12.4	19.5	8	Sewerage and Water Board of New Orleans, Drainage Engineering Department.

Records of Storms from Which Information Has Been Obtained

Of the four records of storms from 1907 to 1920, only one could be used, owing to the fact that some of the gauges were out of order and closed isodyhyetals could not be drawn. During this one storm (April 9, 1920), all the six gauges of the Sewerage and Water Board were in operation. The amounts and rates of precipitation during this storm for selected periods are shown in Table 7.

TABLE 7.—Precipitation During Selected Periods for Storm of April 9, 1920, at New Orleans, La., as Recorded by Automatic Rain Gauges.

(Location of Gauges, Precipitation, in Inches, and Rate of Precipitation in Inches per Hour.)

Time		Period of time, in minutes.	Storm reference designation.	Dublin.		Park.		Jefferson.		City Hall.		London.		Algiers.	
From.	To.			Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.	Inches.	Inches per hour.
12:45	1:00 A.M.	15	*a*	0.48	1.92	0.40	1.60	0.87	3.48	1.38	5.52	0.17	0.68	0.88	3.52
5:35	5:50 A.M.	15	*b*	0.28	1.12	0	0	0.35	1.40	0.70	2.80	0.25	1.00	0.42	1.68
12:30	1:00 A.M	30	*c*	1.26	2.52	1.29	2.58	1.59	3.18	2.29	4.58	0.37	0.74	1.95	3.90
5:30	6:00 A.M	30	*d*	0.55	1.10	0.20	0.40	0.77	1.54	1.28	2.56	0.41	0.82	0.93	1.86
12:15	1:00 A.M.	45	*e*	1.60	2.13	1.37	1.83	2.17	2.89	3.12	4.17	1.23	1.64	2.80	3.73
12:00	1:00 A.M.	60	*f*	2.03	2.03	1.37	1.37	2.82	2.82	3.67	3.67	1.27	1.27	3.45	3.45

In all cases, the records of automatic gauges were corrected by the stick method which is assumed to give the correct total rainfall. If the automatic gauge was high or low, a percentage correction was made proportionately through the period covered by the stick measurement.

Methods of Analysis.—In analyzing these chart records, the method was the same as that followed for the Boston records. The isodyhyetals are shown in Figs. 5 to 10. The method used in computing the average depth of precipitation over the area covered by the gauges was the same as that described for the Boston records. The average rate of precipitation, the area covered at this rate, both in acres and in square miles, the maximum rate of precipitation, and the ratio of the average rate to the maximum rate are shown in Table 8.

A comparison of the average rates of precipitation for durations of 15, 30, 45, and 60 min., respectively, and the area covered is shown in Fig. 11. The frequency of the New Orleans downpours as shown on Fig. 11 was estimated from curves* prepared in 1918 by E. P. Burke, Assoc. M. Am. Soc. C. E.

* "Relation Between Frequency and Intensity of Rainfall Storms of Various Durations, New Orleans, La., 1896–1916."

FIG. 5.

FIG. 6.

FIG. 7.

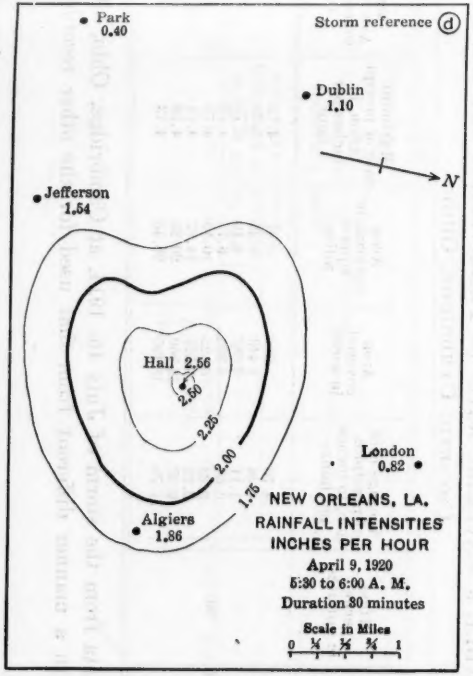

FIG. 8.

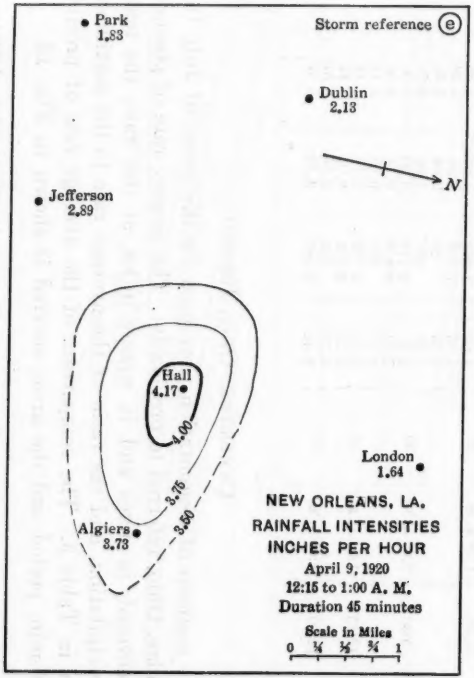

FIG. 9.

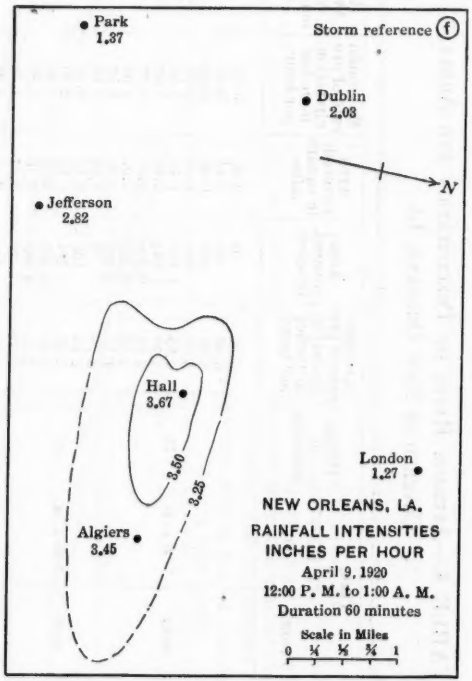

FIG. 10.

TABLE 8.—AVERAGE RATES OF PRECIPITATION AND AREAS COVERED IN NEW ORLEANS, LA.

Date.	Time. From:	Time. To:	Period, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate to maximum rate.
April 9, 1920	12:45	1:00 A. M.	15	5.51	6	0.01	5.52	1.00
				5.38	90	0.14	5.52	0.98
				5.25	307	0.48	5.52	0.95
				5.05	640	1.00	5.52	0.91
				4.85	1 108	1.73	5.52	0.88
				4.70	1 684	2.63	5.52	0.85
				4.53	2 408	3.76	5.52	0.82
				4.35	3 245	5.07	5.52	0.79
				4.17	4 295	6.71	5.52	0.76
April 9, 1920	5:35	5:50 A. M.	15	2.78	13	0.02	2.80	0.99
				2.63	365	0.57	2.80	0.94
				2.45	1 267	1.98	2.80	0.88
				2.28	2 605	4.07	2.80	0.81
				2.11	4 545	7.10	2.80	0.75
April 9, 1920	12:30	1:00 A. M.	30	4.54	26	0.04	4.58	0.99
				4.37	410	0.64	4.58	0.95
				4.20	1 267	1.98	4.58	0.92
				4.13	1 792	2.80	4.58	0.90
April 9, 1920	5:30	6:00 A. M.	30	2.53	26	0.04	2.56	0.99
				2.38	666	1.04	2.56	0.93
				2.21	2 112	3.30	2.56	0.86
				2.03	4 328	6.76	2.56	0.79
April 9, 1920	12:15	1:00 A. M.	45	4.09	205	0.32	4.17	0.98
				3.91	1 133	1.77	4.17	0.94
				3.75	2 502	3.91	4.17	0.90
April 9, 1920	12:00	1:00 A. M.	60	3.59	435	0.68	3.67	0.98
				3.42	2 280	3.56	3.67	0.93

CAMBRIDGE, OHIO, RECORD

Similar methods of computation were used for the storm of July 16, 1914, at Cambridge, Ohio, referred to previously. The average rate of precipitation, the area covered, in acres and in square miles, at this rate, the maximum rate of precipitation, and the ratio of the average rate to the maximum rate, are shown in Table 9. The comparison of the average rate of precipitation for the 90-min. period and the areas covered is shown in Fig. 12.

TABLE 9.—AVERAGE RATES OF PRECIPITATION AND AREAS COVERED, CAMBRIDGE, OHIO.

Date.	Time period, in minutes.	Average rate of precipitation, in inches per hour.	Area covered, in acres.	Area covered, in square miles.	Maximum rate of precipitation, in inches per hour.	Ratio: Average rate to maximum rate.
July 16, 1914	90	4.70	518	0.81	4.73	0.99
		4.46	1 440	2.25	4.73	0.94
		4.11	2 682	4.19	4.73	0.87
		3.68	4 320	6.75	4.73	0.78
		3.23	6 522	10.19	4.73	0.68
		2.73	9 517	14.87	4.73	0.58
		2.27	12 960	20.25	4.73	0.48
		1.76	18 195	28.43	4.73	0.37

The data from the storm of July 16, 1914, at Cambridge, Ohio, although obtained in a manner different from that used for the other records, is of

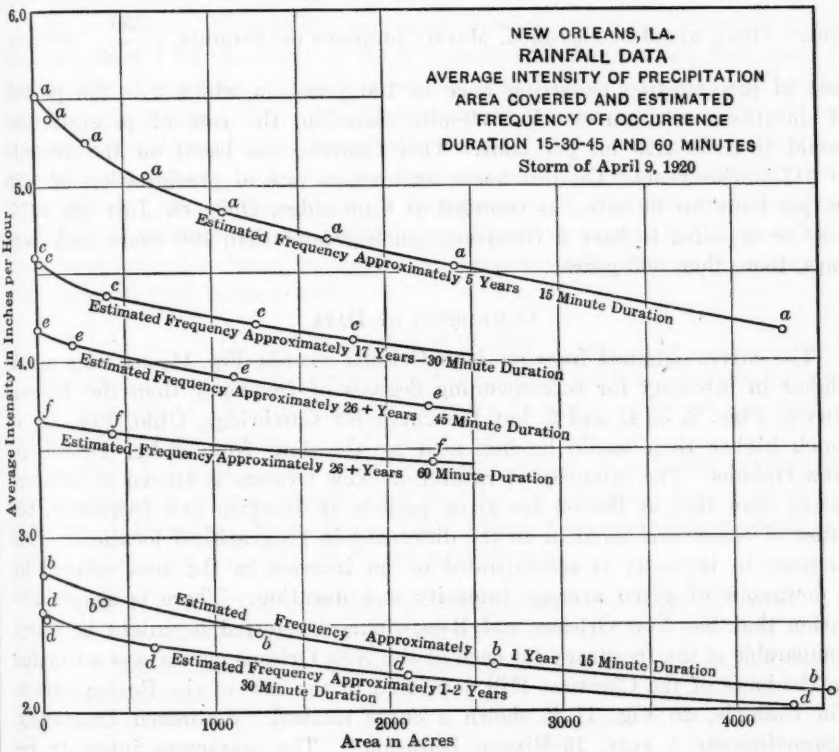
NEW ORLEANS, LA.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AREA COVERED AND ESTIMATED
FREQUENCY OF OCCURRENCE
DURATION 15-30-45 AND 60 MINUTES
Storm of April 9, 1920
Estimated Frequency Approximately 5 Years 15 Minute Duration
Estimated Frequency Approximately 17 Years—30 Minute Duration
Estimated Frequency Approximately 26 + Years 45 Minute Duration
Estimated Frequency Approximately 26 + Years 60 Minute Duration
Estimated Frequency Approximately 1 Year 15 Minute Duration
Estimated Frequency Approximately 1-2 Years 30 Minute Duration
Average Intensity in Inches per Hour
Area in Acres
6.0
5.0
4.0
3.0
2.0
0
1000
2000
3000
4000

FIG. 11.

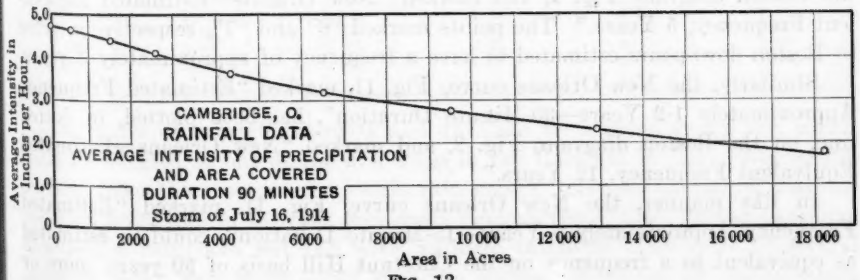
CAMBRIDGE, O.
RAINFALL DATA
AVERAGE INTENSITY OF PRECIPITATION
AND AREA COVERED
DURATION 90 MINUTES
Storm of July 16, 1914
Average Intensity in Inches per Hour
Area in Acres
5.0
4.0
3.0
2.0
1.0
0
2000
4000
6000
8000
10 000
12 000
14 000
16 000
18 000

FIG. 12.

interest because of the great intensity for a duration of 90 min. and the great areas covered. Few storms of equal or greater intensity and duration have been recorded in the United States. For Group 3 Stations, in which Cambridge, Ohio, would be classified, Meyer* proposes the formula, $\frac{256}{t + 25}$, for the rate of precipitation occurring once in 100 years, in which t is the period of duration, in minutes. For 90-min. duration, the rate of precipitation would then be 2.23 in. per hour. This formula was based on the records of 317 station-years. On this basis, an average rate of precipitation of 4.73 in. per hour for 90 min., as recorded at Cambridge, Ohio, on July 16, 1914, may be expected to have a frequency much greater than 100 years and, perhaps, more than 300 years.

Comparison of Data

The curves obtained from the New Orleans records, Fig. 11, are very much higher in intensity for corresponding degrees of frequency than the Boston curves, Figs. 2, 3, 4, and 5, but the curve for Cambridge, Ohio, Fig. 12, is much higher than would be indicated by the data from either Boston or New Orleans. The intensity of rainfall at New Orleans is known to be more severe than that at Boston for given periods of duration and frequency, because of conditions incident to the difference in geographical location. This increase in intensity is accompanied by an increase in the area covered by a downpour of given average intensity and duration. There is some indication that the New Orleans and Boston curves referred to, might be fairly comparable if the frequency for each of the New Orleans curves was estimated on the basis of the Chestnut Hill data used in the case of the Boston curves. For example, on Fig. 11 is shown a curve marked, "Estimated Frequency, Approximately 1 year, 15-Minute Duration." The maximum intensity recorded for this downpour, *b*, is given in Table 7 as 2.80 in. per hour (City Hall gauge). On the basis of the Chestnut Hill data (Table 5), such an intensity for a duration of 15 min. is estimated to have a frequency of approximately 5 years. This New Orleans curve is shown as a dotted line on the Boston diagram, Fig. 1, and marked "New Orleans—Estimated Equivalent Frequency, 5 Years." The points marked "6" and "7", respectively, refer to Boston downpours estimated to have a frequency of approximately 3 years.

Similarly, the New Orleans curve, Fig. 11, marked "Estimated Frequency Approximately 1-2 Years—30-Minute Duration", has been plotted, in dotted line, on the Boston diagram, Fig. 2, and marked "New Orleans—Estimated Equivalent Frequency, 12 Years."

In like manner, the New Orleans curve, Fig. 11, marked "Estimated Frequency Approximately 5 Years—15-Minute Duration", could be estimated as equivalent to a frequency on the Chestnut Hill basis of 50 years, more or less.

The Cambridge, Ohio, storm is of high intensity and the curve from this storm, Fig. 12, can only be compared with the other curves in a general way.

* "Elements of Hydrology", p. 183.

The frequency is probably in excess of 100 years. The curve has the same general form as the 60-min. duration curves in Fig. 4.

The interesting point is the similarity between the New Orleans and the Boston curves. On the basis of the Chestnut Hill frequencies, they are approximately comparable. If such a relation could be established, with ample data for a foundation, it would add considerably to the information available for use in Boston and other comparable districts. The probability of obtaining sufficient records in Boston for frequencies of 15 years, or greater, with the number of gauges now in use, is doubtful, but if it should prove possible to use New Orleans records properly translated in terms of Chestnut Hill frequencies, the deficiency in the Boston records could thereby be made up. This is worthy of further study.

Practical Use of Data

As stated previously, it is not intended to draw final conclusions from so few measurements. It is of interest, however, to study a method for making practical use of such data.

On Figs. 2, 3, and 4, lines have been interpolated, showing the relation of average intensity of precipitation and area for an estimated frequency of occurrence of 10 years. From these, Table 10 has been compiled, showing the same relations for durations of 30, 45, and 60 min.

Table 10.—Average Intensity of Precipitation and Area Covered, Boston, Mass. Estimated Frequency of Occurrence, 10 Years.

Duration, 30 Min.		Duration, 45 Min.		Duration, 60 Min.	
Area, in acres.	Average intensity of precipitation, in inches per hour.	Area, in acres.	Average intensity of precipitation, in inches per hour.	Area, in acres.	Average intensity of precipitation, in inches per hour.
0	2.48	0	1.94	0	1.60
500	2.34	500	1.85	500	1.55
1 000	2.26	1 000	1.80	1 000	1.52
1 500	2.20	1 500	1.76	1 500	1.49
2 000	2.15	2 000	1.74	2 000	1.47
3 000	2.07	3 000	1.69	3 000	1.43
4 000	1.98	4 000	1.65	4 000	1.40
5 000	1.92	5 000	1.61	5 000	1.38

The curve of rainfall intensity and duration for a given frequency, which must be selected as one of the first steps in the Rational Method, is generally derived from the records of a single rain gauge. Its use, therefore, may involve an error due to the fact previously demonstrated, that, during excessive downpours, rain does not fall with uniform intensity over areas of several hundred acres and larger, and that the average intensity of precipitation over such areas is appreciably less than the maximum intensity indicated by one gauge. Theoretically, the rate of precipitation should be reduced with the increase in area, regardless of its size.

Although the records as obtained from a single gauge do not always record the peak of the storm, they do represent the frequency with which given in-

tensities have occurred at the gauge. A rainfall intensity curve that is compiled from records covering a considerable period is probably influenced to a greater extent by the storm peaks than would be the case if the record covered only a short period.

The variation in intensity is well illustrated by Table 10, which shows that for areas of 1 000 acres and larger, and more particularly with the shorter durations, the average intensity is somewhat less than the maximum intensity.

For the 30-min. duration, the difference between the maximum intensity (average intensity at zero area) and the average intensity of precipitation for an area of 1 000 acres is only 0.22 in. per hour, which is less than 9 per cent. Similarly, for the 45-min. duration, the difference is 7.4%, and for the 60-min. duration, the difference is 5 per cent. Such differences as these are too small to consider, in view of other errors involved in all rainfall studies. With larger areas the difference becomes greater and may be taken into account. If such a reduction is to be made, tables or diagrams can be prepared to facilitate the work.

With long narrow drainage areas, the flow in the sewers is affected considerably by the direction of movement of a storm. Such a condition is described by Mr. Horner in referring to the storm of August 24, 1918, at St. Louis, mentioned previously. The storm moved in the direction of flow through the sewer mains and at about the same velocity, causing a greater flow in the sewers than might have been the case had the path of the storm been at right angles to the general direction of the sewer system. In using the average intensity of precipitation in design, it is assumed in every case, that the average intensity of precipitation occurs over the whole tributary area regardless of its shape, thereby approximating the condition that will give the greatest run-off for the assumed intensity, duration, and frequency, without introducing a greater factor of safety, which would be the case if the maximum intensity is used.

Conclusions

The tables and diagrams in this paper present definite facts on the relation between intensity of rainfall and areas covered for excessive rates of precipitation with varying periods of duration and degrees of frequency. These data are not sufficiently comprehensive to warrant final conclusions. They may serve, however, as an aid to judgment, if proper reservations are kept in mind. It is hoped that the presentation of these facts with the accompanying comments may stimulate the making of further measurements, not only in Boston, but in such cities as New Orleans, St. Louis, Washington, D. C., and New York, where a fair beginning has already been made in establishing recording rain gauges. Records from New Orleans would be particularly helpful, because of the frequency of intense storms and the probability of obtaining data that would be of value elsewhere. Excessive rain storms having a frequency of 5 years or greater, and durations of 15 to 120 min., are of particular interest.

Information is needed in order to determine the effect on the records of excessive precipitation, of the height of a recording rain-gauge collector

above the surface of the ground. It is believed at present, that the height of the collector (or lack of wind guards), has less effect on the records of excessive downpours than on those of moderate rainfall, because with the latter, the wind may act more effectively on the rain-drops, tending to blow them past the collector. During excessive downpours, the rain-drops fall more nearly perpendicularly.

One fundamental assumption of the Rational Method of design is that the rate of precipitation is uniform throughout the tributary area under consideration. The data presented herein indicate that, for tributary areas of the size ordinarily considered in the design of storm-water drains, the variation in the rate of precipitation within the area is so small that the fundamental assumption of a uniform rate (Rational Method) appears to be justified. These data also indicate that it may be practical with very large tributary areas to make a reduction in the intensity curve, which is ordinarily based on the duration of the storm irrespective of the area covered by it.

A comparison of the New Orleans and Boston data indicates the possibility of using observations on the distribution of intense rainfall at New Orleans in studies for Boston, or other comparable districts, by properly translating the estimated frequencies of occurrence of given intensities and durations. Similarly, it is possible that records from other cities may be compared. If this tentative conclusion is finally substantiated, the practical value of the accumulated records will be greatly increased.

Attention is directed to the paucity of information on the run-off from sewered areas of various degrees of imperviousness. Attempts have been made to collect such information from large areas, but the efforts have generally been attended by serious difficulties, and the results have not been as helpful as was hoped.

The values that should be allowed for inlet time, need to be substantiated by additional observations. Engineers are called on more and more, to design drains and sewers to relieve flooding from storm water, particularly in the densely built-up sections of the larger cities. The demands of the public and of property owners are increasing, and these demands require more careful design of the necessary drainage structures. To aid judgment in meeting these problems, efforts are being made and should be extended in order to increase knowledge along the lines described.

The chart records of the several gauges were supplied the writer by officials in charge, and other assistance was rendered. For this help, the writer's appreciation is due in particular to Henry A. Varney, M. Am. Soc. C. E., Town Engineer, Brookline, Mass.; Lewis M. Hastings, City Engineer, Cambridge, Mass.; Edgar S. Dorr, M. Am. Soc. C. E., Office Engineer, Public Works Department, Frederick A. Lovejoy, Engineer of Design, and William F. Hughes of the Sewer Service, Public Works Department, Boston, Mass.; William E. Foss, M. Am. Soc. C. E., Director and Chief Engineer, Samuel E. Killam, Superintendent of Distribution Section, and William E. Whittaker, Chief Clerk, Water Division, Metropolitan District Commission; William L. Vennard, City Engineer, Lynn, Mass; Lewis A. Wells, Chief Observer, Blue Hill Meteorological Observatory, Milton, Mass.; Charles F. Marvin, Chief

and J. W. Smith, Meteorologist, U. S. Weather Bureau; and George G. Earl, M. Am. Soc. C. E., General Superintendent and Chief Engineer, and Alfred F. Theard, M. Am. Soc. C. E., Principal Assistant Engineer, Sewerage and Water Board, New Orleans, La. The writer is also under obligations to his partners, Leonard Metcalf, Harrison P. Eddy, Charles W. Sherman, and Almon L. Fales, Members, Am. Soc. C. E., and to the members of the staff of the firm of Metcalf and Eddy, particularly Mr. Scott Keith, for valuable assistance in carrying on the investigations and in preparing the paper.

THE DISINTEGRATION OF CEMENT IN SEA WATER

Discussion*

By Messrs. John Stephen Sewell, J. A. Kitts, and William G. Atwood and A. A. Johnson.†

John Stephen Sewell,‡ M. Am. Soc. C. E. (by letter).§—Every one interested in the use of cement and concrete should feel indebted to the authors for having assembled so much accurate and reliable information scattered through such a variety of publications and over such a length of time.

By reading the paper, and the discussions thereon, the conclusion is inevitable that present-day cements, when exposed to sea water or to water containing alkali, are in serious danger of disintegration. The conclusion seems also to be justified that the sources of these dangers are largely determined; also, that practical methods of curing the troubles are at least indicated, if not perfected. In view of the importance of the question from the standpoint of the integrity and permanence of invested capital, it is to be hoped that the work of the Committee on Marine Piling Investigations may be carried forward without interruption to final conclusion.

The discussions indicate a belief on the part of some competent authorities that with sufficient precaution in the selection of material, making, and placing of the concrete, present standard cements will give reliable and satisfactory results. After reading what is said by those engineers who believe that present cements can be made to give satisfactory results in sea water, and in alkaline water, the writer suggests that there are too many precautions to be observed. One of the things that has made the United States great and rich has been the success with which Americans have multiplied man power, and have designed structures and prepared materials so that the minimum of skill and labor is everywhere required in the work of erection and installation. If there were no indications whatever of possible changes in the character of cement, that would make it more durable under the conditions considered, then one would be justified in stating that all the elaborate precautions necessary to give present standard cements a reasonable life, must be observed. However, in view of the fact that a solution of the problem in the form of making the cement itself chemically immune, is plainly indicated, it is the judgment of the writer that the best policy is to devote a reasonable amount of time, attention, and money to the problems of making and producing on a commercial scale, a cement which will be chemically permanent whether or not it is placed with great skill.

* Discussion on the paper by William G. Atwood and A. A. Johnson, Members, Am. Soc. C. E., continued from December, 1923, *Proceedings*.

† Authors' closure.

‡ Pres., Alabama Marble Co., Birmingham, Ala.

§ Received by the Secretary, July 2, 1923.

J. A. KITTS,* ASSOC. M. AM. SOC. C. E. (by letter).†—In making a study of the action of sea water on cement concrete, it is important that observations be made to distinguish between mechanical disintegration of the concrete, and chemical decomposition of the cement, determining the relative measure of each.

It is the writer's observation that, generally, the disintegration of American, standard Portland cement concrete in sea water is due to: (1) lack of cement; (2) excess of minute, and otherwise unsuitable, fine aggregate; (3) coarse aggregate of porous, soft, or otherwise unsuitable rock, having its maximum size limited to an unwarranted extent; (4) excessive mixing water; and (5) lack of constant moisture in curing. All these conditions increase the permeability of the concrete.

No doubt, one cement may have more resistance to the chemical action of sea water than another. However, tests such as those made under the direction of H. Burchartz,‡ in which arbitrary proportions of 1 : 2 : 6 and 1 : 4 : 10, are used, are not safe criteria by which to judge whether a cement would be suitable for use in sea water if the concrete were properly proportioned. Under certain conditions the 1 : 2 : 6 mixture may be ideal. The 1 : 4 : 10 mixture, however, will certainly be highly permeable.

It appears to be a proper assumption that the more permeable a concrete mixture, the more rapid will be its chemical and mechanical reduction in sea water.

Impermeable concrete may be obtained by observing the following precautions: (1) selection of impermeable aggregates; (2) formation of a dense, coarse aggregate by grading to the maximum size which may be used; (3) reduction of voids in the coarse aggregate by adding a proper proportion of graded fine aggregate (an excess will increase the voids); (4) use (as a minimum) of a volume of cement equal to the volume of voids in the mixed aggregate; (5) use of the minimum quantity of tempering water necessary to secure a plastic mixture or the initial flowing consistency (an excess will increase the voids); and (6) the curing process conducted under continuously moist and temperate conditions to complete the hydration of the cement.

A very fine sand should not be used in sea water concrete. Particles approximating the size of the cement are merely an adulteration, and decrease the strength and density of mortar.

The disagreement of the results of the various investigations mentioned by the authors, are due probably to the properties of the aggregate mixtures in which the various types of cements have been used, and the ratio of cement to aggregate.

The following notes, covering a special study of concrete for sea water, may be of interest: In 1914, the writer made tests of available aggregates to determine the most suitable mixture for use in armoring the Colon Harbor Breakwaters, Panama Canal. The only objects in these tests were to secure an

* Field Engr., Am. Steel & Wire Co., San Francisco, Calif.

† Received by the Secretary, December 3, 1923.

‡ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1046.

impermeable concrete. As a result, the proportioning was fixed as 1 part cement to 4.5 parts Chagres River gravel (Gamboa).

The cement was a standard brand used throughout the Canal project. Chagres River gravel (bank run) was used as found, without any screening or re-combination processes. It had the following characteristics (from memory):

Size	0 to 6 in. in diameter; largely igneous rock fragments.
Percentage of voids	16 to 21.
Specific gravity	2.65.
Shape	Rounded.
Absorption	Low (minimum of all available aggregates).
Silt	Negligible.
Organic matter	Negligible.
Soundness	Particles hard and tough.

The maximum porosity found in any laboratory was 21% of voids, and the ratio of cement to voids in the aggregate varied, therefore, from 1.01 to 1.39. Tests of this aggregate were also made by L. C. Sabin, M. Am. Soc. C. E.

Although the mixture was unusually rich in cement in comparison with that used for mass concrete elsewhere on the Canal project, the use of bank-run gravel without screening or re-combining processes and the handling of one aggregate instead of two, effected an economy comparable with the excess cost of the cement. The tests are a matter of record, and the concrete masses, varying in size from 1 cu. m. to 25 tons, have been subjected to wave action since 1915.

This work was carried out under the direction of Rear-Admiral H. H. Rousseau, C. E. C., U. S. N., M. Am. Soc. C. E., as Division Engineer of the Division of Terminal Construction, Panama Canal.

It is hoped that the authors will supplement this valuable paper with the final results of their studies.

WILLIAM G. ATWOOD* and A. A. JOHNSON,† MEMBERS, AM. SOC. C. E. (by letter).‡—The writers have been much gratified by the interest shown in their paper and appreciate the addition to the knowledge of the subject which these discussions furnish. It is especially gratifying to note the almost universal agreement with which their ideas and conclusions have been received.

It was impossible, within the limits of the paper, to consider the economic side of the question of disintegration suggested by Mr. Nathan C. Johnson.§ Some concrete structures may disintegrate and still fully serve the purpose for which they were constructed, since obsolescence or other cause may make it unnecessary for a structure to endure for more than a comparatively few years, in order to fulfill its purpose. Generally, concrete structures are

* Director, Committee on Marine Piling Investigations, National Research Council, New York, N. Y.

† Asst. to Director, Committee on Marine Piling Investigations, National Research Council, New York, N. Y.

‡ Received by the Secretary, December 6, 1923.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1647.

intended to be "permanent", although, actually, this term is relative and not absolute, and any disintegration, especially if it is progressive, is of extreme importance, as it will cause either heavy maintenance charges or failure, or both, before the expiration of the expected life of the structure.

The report of the Institution of Civil Engineers of 1920 with the Interim Reports of 1921 and 1922, mentioned by Mr. Moyer,* indeed offers much valuable information. Unfortunately, as is the case in the United States, construction records were generally incomplete, but for use in a forthcoming report of the Committee on Marine Piling Investigations a study was made of the available construction data in the foregoing three reports, as well as in the reports submitted to the International Navigation Congress in 1908, 1912, and 1923, and in reports from other foreign sources. Following this study, all structures built before 1912, for which reasonably definite construction data were available, were listed. Thirty such structures were found, of which the oldest was built in 1855, and twenty-four since 1880. Of the thirty structures inspected, —twenty-seven within the last three years—seventeen showed deterioration. Of the thirteen which showed no disintegration, one had a heavy granite facing and four contained admixtures of trass or pozzuolana. Eight sound structures in thirty is not an especially good record for a "permanent" material.

Service records of structures in the coastal harbors of the United States have been assembled in the same manner; in this latter case, structures of more recent date have been included in order that the records of their construction might be available for future investigators. For commercial and other reasons, it has been impossible to secure a list of structures that was at all complete. Including only those structures for which could be secured reasonably complete construction and recent inspection data that the owners were willing to have published, this list is as follows:

Constructed	1856.....	1 structure	Good condition.
"	1880–89....	2 structures....	One good, one deteriorated.
"	1890–99....	2 "	Both deteriorated.
"	1900–09....	10 "	Three good, seven deteriorated.
"	1910–14....	14 "	Six good, eight deteriorated.
"	1915–19....	12 "	Nine good, three deteriorated

It is true that this record includes only a small number of structures, but they are located from Maine to Puget Sound, and although the construction records in a few cases show probable causes of deterioration other than chemical attack, most of them do not.

The reports of the San Francisco Bay Marine Piling Committee of 1920, 1921, and 1922, contain a number of tables and much valuable information on this subject. One table gives a record of 5 198 reinforced cylinder supports in 12 structures built between 1909 and 1918, of which 457 had required repairs before 1922. These cylinders were cast in place in open caissons which had been unwatered.

Three structures in which reinforced piles were used, are reported in detail. The first of these, constructed in 1911, contained 135 piles of which 41, in

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1919.

1922, showed cracks over the reinforcement above high water; the second, constructed in 1912, contained 141 piles of which 36 were cracked similarly, and the third, built in 1915, contained 2 492 piles of which 56 were badly, and 215 slightly, cracked in the same location.

Several discussions* notably those of Messrs. Flinn, Wiggin, Allen, and Riedel, emphasize the need for a cement that will not disintegrate under attack by sulphate-bearing water in locations other than in salt water and in alkali districts, and this fact accentuates the necessity for a thorough investigation of the whole problem.

Several of the structures mentioned by Gen. Black† have been inspected since the receipt of his discussion, under the direction of the officers of the Corps of Engineers, with the following results: Repair work at Fort Marion, Fla., was done in 1886-87, all above high water. Beach sand, coquina gravel, and European Portland cement, in the proportion of 1 : 4 : 6.5, and Rosendale cement, in the proportion of 1 : 2 : 4, were used. This work is still in good condition.

The wall at Fort Taylor is subject to wave action as well as exposure to salt water. It was built in 1856 using the F. O. Norton brand of Rosendale cement and crusher-run limestone in the proportions of 1 : 7. Part of the wall was destroyed by the hurricane of 1910, but the remainder is sound. The concrete is soft, but no softer than the aggregate of which it is made.

A part of the groins have been buried in sand for many years, but those on Anastasia Island are still exposed. Groin No. 1, built in 1889, has concrete capping of blocks about 2 to 2½ by 5 by 6 ft. A poor grade of beach sand and coquina gravel formed the aggregate. About nine-tenths of the blocks were made of Anchor brand Portland cement in the proportions of 1 : 4 : 6.5; also, a small quantity of Sphinx and Dyckerhoff brands was used in the same proportions. Other blocks were made of Rosendale cement and of the Hydraulic Lime of Teil (France), in the proportions of 1 : 2 : 4. The concrete is somewhat deteriorated, but is all in fair condition, and although the records as to the location of the blocks made with the different cements cannot be found, the disintegration is so uniform that there is no information as to the relative value of the different cements.

Groin No. 4, built in 1890-91, has rectangular blocks, 2.8 by 4 by 4 ft., made of Saylor's brand cement with the same aggregates as those previously mentioned, in the proportions of 1 : 3 : 5 and 1 : 2 : 4. The blocks have suffered some deterioration.

In reply to Mr. Wiggin's‡ request, it should be said that, in the preparation of the paper, all available information regarding experiments was considered, and it was the intention to respect all viewpoints. The conflicting results of these German experiments were due in part to the fact that the tests were not performed on a strictly comparable basis, as originally stated.

* *Proceedings*, Am. Soc. C. E., September, 1923, pp. 1649, 1652, 1655, and 1657, respectively.

† *Loc. cit.*, p. 1648.

‡ *Loc. cit.*, p. 1652.

Mr. Allen* calls attention to the opinion of Le Châtelier regarding the desirability of a low alumina content in cement exposed to salt water. Many other investigators concur in this opinion of Le Châtelier, but the opinion applies only to Portland cement in which the principal binding elements are the various calcium silicates, whereas, in the *ciment fondu,* the binding elements are calcium aluminates and the silicates are secondary constituents. The two cements are entirely different, and a constituent which might be a deleterious element in one would have an entirely different effect in the other.

Professor White's reply to his own question† as to the reason for the greater disintegration within the tidal range than below low water, is believed to be correct. The concentration of salts in the pores or hair cracks is undoubtedly greater than that found below low water, where the normal salt content remains reasonably constant. It would be interesting to determine definitely whether, as seems to be indicated by some experiments, the coefficient of expansion and the consequent cracking are greater with rich than with lean concrete.

M. Jeanneret‡ calls attention to one very important point often overlooked, that the chemical composition of the binding medium after setting is the important thing and not the composition of the cement itself, except in so far as it affects the former. The permeability tests reported are of importance, and many such tests should be made.

Mr. Hiroi's tests§ are of especial interest as showing the tendency of Portland cement not exposed to sea water to regain the strength lost in the third to seventh year and the absence of this recovery in salt water. It is to be hoped that the remaining briquettes of this series were not lost in the recent earthquake and that the 50-year test may be completed.

The error in the location of Aberthaw, pointed out by Mr. Walker,‖ is acknowledged. The new specifications of the British Engineering Standards Association represent a distinct advance over present practice in that there is officially recognized by this important body the fact that binding materials other than Portland cement are of sufficient value to justify the issuance of a specification. These specifications, as stated by Mr. Walker, limit the additional silica to a quantity between 35% and 65% and specify the methods of mixing and grinding. They do not give any information as to the manner of determining the exact quantity of silica to be added between these rather wide limits.

The method of protection for piles described by Mr. Cykler¶ is a very interesting development; fortunately, it is being tested on a scale which will give good service records after the lapse of sufficient time. It is, of course, applicable only to pre-cast piles or other similar members of a size sufficiently small to allow tank treatment. It would appear probable that the cracking of the piles on account of the difference between the coefficients of expansion of the

* *Proceedings,* Am. Soc. C. E., p. 1655.
† *Loc. cit.,* p. 1661.
‡ *Proceedings,* Am. Soc. C. E., October, 1923, p. 1762.
§ *Loc. cit.,* p. 1764.
‖ *Proceedings,* Am. Soc. C. E., November, 1923, p. 1915.
¶ *Loc. cit.,* p. 1916.

concrete and of the steel would result in a reduction of strength, although it is claimed that the cementing strength of the asphalt prevents this. Moreover, it is contended that this cracking has been practically eliminated by a change in the methods of treatment during the last few months.

It is unfortunate that experiments such as those indicated by Mr. Jewett,* have not been made. The experiments which have been carried on in the alkali districts by the U. S. Bureau of Standards, the Drainage and Highway Bureaus of the U. S. Department of Agriculture, and the Portland Cement Association, have produced valuable information with regard to mixtures although as yet they do not seem to have resulted in a concrete which will resist attack over a considerable period.

The writers are indebted to Dr. Grün† for calling their attention to the omission from the paper of reference to the writings of Dr. Hermann Passow. Dr. Passow's articles on blast-furnace slag cement and its ability to resist the attack of sulphate-bearing waters, which appeared in several issues of the *Tonindustriezeitung,* in 1916, were consulted, but were inadvertently omitted from the Bibliography. As pointed out by Dr. Grün, this cement is properly classified with the pozzuolana group and, when properly prepared, enjoys all the advantages belonging to that class of cements.

Mr. Newhall‡ suggests a cause for disintegration different from that presented by the writers. It is entirely possible that carbon dioxide is an active disintegrating agent in spite of the fact that the calcium carbonate film has been thought to give protection. The experiments made by the Carnegie Institution of Washington at the Dry Tortugas Laboratory seem to indicate the probability that carbon dioxide in sea water will dissolve lime. This is one of the many phases of the subject which requires careful scientific study.

The reports of the U. S. Geological Survey do not indicate that it would be difficult to secure the materials for either the high silica or high alumina cement. Volcanic ash, or pozzuolana, is found in a number of locations in the Far West; blast-furnace slag is produced in large quantities in the Central and Eastern areas; and diatomaceous earth is also found in both the East and the West. The material from some if not all of these sources of supply will probably be found to contain soluble silica. Bauxite deposits have been, or are being, worked in a number of the Southern States and may possibly exist elsewhere. It does not appear that Mr. Newhall's fears of insufficient quantity or of prohibitive expense are well founded.

The information contained in the discussion of Señor de Castro§ is exceedingly valuable. This statement by an engineer of long experience and one who has given the problem of construction in salt water much intelligent study, indicates clearly the requirements for a stable cement and how to obtain it. Many of the attempts to use silicious admixtures have been made without determining the free lime content of the cement after setting and the percentage of soluble silica in the material added; the methods of mixing, too, have frequently not been as thorough as those recommended by Señor de Castro.

* *Proceedings,* Am. Soc C. E., November, 1923, p. 1920.
† *Loc. cit.,* p. 1922.
‡ *Proceedings,* Am. Soc. C. E., December, 1923, p. 2060.
§ *Loc. cit.,* p. 2062.

It has been stated by many experimenters, and is generally believed, that a cement with which silica has been mixed, has less strength than it had before the silica was added. It will be noted that Señor de Castro states that the reverse is true if the proportions are correct and the proper method of mixing is used.

A number of interesting and important papers on the general subject of concrete disintegration in sea water were presented at the meeting of the International Navigation Congress in London, England, in August, 1923, and a brief review of the conclusions are thought to be of interest in so far as they apply to the disintegration of concrete and the corrosion of reinforcement.

In discussing the work done at Termonde, Belgium, 1921-22, Mr. R. Derycke, Chief Engineer and Director of Roads and Bridges, and Messrs. Ph. Goossens and J. Haché, Principal Engineers of Roads and Bridges, give many valuable details. With regard to the binding medium, they state, as follows:

"To avoid disaggregation due to the sea water, it was necessary that the concrete be entirely free from free lime. Of the two methods of making clinker cement, that by the cold way consists in mixing with the finely powdered granulated slag a rectifying powder of slaked and sifted lime mixed so that there remains neither anhydrite nor free base after crystallization, which is really the phenomenon of setting.

"For fear that the mixing had not been rigorously kept to proportions, or that in spite of exact proportions, there existed a free base, that is to say, lime in the hardened concrete, the concrete mixture received an excess of silicious anhydrite in the form of slag crushed and granulated.

"The slag, quickly cooled on leaving the blast furnace, acquires hydraulizing properties, that is to say, the silica which it contains is silicious anhydrite which combines when cold with the free lime to form with it lime silicates.

"This amounts then to adding to the mortar pozzuolanic, matters of which the good effect on the preservation of concrete exposed to sea water is sufficiently well known.

"As a matter of fact, the granulated slag, finely crushed, forms a cement with an insufficient proportion of lime. The cement which serves as binder for the concrete of the quay walls at Termonde, is, therefore, a distinctly acid cement.

"The choice of the slag has a great importance on the setting and inchangeability of the crystallized mass. Black slag is unsuitable for use, because of the great proportion of metallic oxides. Reddish yellow slag is to be avoided, its hydraulizing properties being very weak. Clear yellow, pale yellow, or bright yellow slag is also to be considered as unsuitable, its hydraulicity being insufficient. The grayish-blue slag is the best, because of its remarkable hydraulizing qualities.

"The setting of slag cement is generally very slow, slower than with slow-setting Portland cement."

The cost of concrete made with this cement is said to have been 70 francs per cu. m., as compared to 93 francs for equivalent concrete made with Portland cement.

In their report, Messrs. J. A. Ringers, Chief Engineer of the "Rijkswaterstaat", and C. Tellegen, Engineer of the "Rijkswaterstaat", quote from and agree with the conclusions of Mr. Blocq van Kuffeler presented in 1912. These conclusions call attention to the necessity for tight forms and thorough mixing; they recommend that trass be used and that, wherever possible, the concrete be thoroughly seasoned before being immersed in salt water. The writers

add four other conclusions which recommend the use of plastic as against rammed concrete; the use of a rich mixture to increase impermeability is also said to be desirable, but attention is called to the fact that increase in the cement content increases contraction and consequent cracking; further research is recommended with the view of determining which cements are best for harbor structures; the cover of 3 cm. to 4 cm. for reinforcement is recommended as the minimum for use with the rich mixtures customary in Holland.

Señor Manuel Maese, Engineer of the Port of Valencia, Spain, in his report on the works in that port, states, in part, as follows:

"The decomposition of the concrete in contact with the sea water occasions nasty accidents, more or less serious. In Spain such cases have happened in several ports; for example, in that of Vigo where the artificial blocks of the enclosure wall of a mole have rapidly decomposed, causing the destruction of the work.

"It is for this reason that engineers are seriously occupying themselves with the question of the maintenance of concrete in maritime works. Some believe that the decomposition of this material must happen sooner or later; others that if suitably employed it merits complete confidence, but all are agreed as to the need for the most minute precautions in the use of concrete, and especially in that of not putting it into contact with sea water until as long as possible after complete hardening.

* * * * * * * *

"To avoid as far as possible the decomposition of the concrete and mortar in the sea water, the cement will be mixed with about one-third of its weight of finely pulverized puzzolane, which will fix in a stable combination, owing to the silica it contains, the hydrate of lime which the cement may contain and that which setting frees, to which the decomposition is attributed. Moreover, the puzzolane, by reducing the voids in the concrete and mortar, renders these more water-tight and improves their conditions of subsistence in the sea. For the same reason, thick-grained sand only will be used (from 0.002 m. to 0.006 m.), the quantity of grains exceeding 0.004 m. must be at least 40 p. c. of the whole, although its cost is high: 22 pesetas per cu. m.

"The concrete used for the hollow blocks will be composed of 350 kg. of Portland cement and 120 kg. of puzzolane per 400 cu. decimeters of sand and 800 of crushed stone of from 0.01 m. to 0.10 m., and of which the volume of pieces of 0.05 m. to 0.10 m., shall be double that of the small stone."

Señor de Castro, Engineer of Roads, Canals, and Ports of Spain, describes a concrete failure in a breakwater where, in addition to the usual disintegration near low-water level, concrete deposited in bags 12 m. below low tide also failed in five years. In this case, the Portland cement passed all specifications and the blocks placed above low water were seasoned for 90 days. An analysis of the substance which replaced the cement was as follows:

Lime	13.4%
Magnesia	12.9%
Alumina	8.5%
Silica and sand	35.0%
Carbonic acid	22.2%
Sulphuric acid	4.6%
Chlorine	2.0%
Iron oxide	2.1%
Total	100.7%

Mr. A. Poulsen, Chief Engineer of the Maritime Works of Denmark, reports in detail a series of tests of both mortar and concrete, commenced in 1896. His summary of the results is, as follows:

"After a period of 20 years in slightly salt sea water (½%), the mortar cubes have become damaged; more so in sea water with higher percentage of salt (3 to 3½% of salt). The richest mortars are the most durable. The daily tidal change is remarkably damaging, and the more so because the frost cracks the mortar which is decomposed by the salt. The fine natural sand, although its mechanical resistance is a little inferior to the 'normal sand', does not cede to it regarding its durability in the sea."

Mr. Poulsen follows this with a statement as to the present practice in Denmark, as follows:

"Since 1910, owing to the unfavorable results of the dikes on the west coast, and after the results of the experiments named above, which were already instructive after ten years, and which were published at the Fifth Congress of the International Association for Testing Materials (Copenhagen, 1909), the Direction of Maritime Works was of opinion that Portland cement for concrete in sea water must be mixed with puzzolana.

"We have used during a few years two kinds of puzzolanas: The Rhenish Trass, recognized for a long time as very proper for maritime works, and a quite new puzzolana, the 'Mo-Ler' of Denmark, a diatomaceous earth, which is found in the islands west of Limfjord (Jutland N.), of which I had discovered the excellent qualities requisite for a puzzolana.

"The qualities having been verified in practice for some years, we have applied only the 'Mo-Ler' under the patented form of 'Cement Mo-Ler', manufactured at the works of the 'Aalborg' Portland Cement Company. Of this, up to the present time, 15 000 tons (of 1 000 kg.) for the dikes of Thyborön have been manufactured, and it has also been used during late years for the large North Sea ports which are in course of construction (*e. g.*, Hirtshals). Thanks to the plasticity of the 'Mo-Ler' mortar, ordinary concrete caissons have been made of it for placing below the level of the sea, instead of making these caissons as they were projected of Kinniple concrete (*i. e.*, waste stone set in cement, without sand), a rather good method, but more expensive. At Thyborön, we have also made some reinforced concrete with Mo-Ler Cement."

M. Voisin, Inspector General of Roads and Bridges of France, reports the construction of two identical quays of reinforced concrete at Port-en-Bessin, one with Portland cement and the other with *ciment fondu*. These structures will yield in time valuable information as to the comparative value of these two binding media.

Two lines of study to prevent the corrosion of reinforcement are evident in these papers, one the attempt by the use of silicious admixtures or rich concretes to obtain an impervious material, the other by the development of a metal for reinforcement which will not corrode. The exhaustive experiments being carried on under the direction of the Institution of Civil Engineers are described in a report by Sir Robert Hadfield.

There is no uncertainty in the minds of the writers as to the necessity for a thorough study of Portland cement from the time the material is quarried until the concrete is in the forms, and especially regarding the methods of manufacture. The fact that some of the oldest structures built with "Natural cements", or Portland cement made by processes which have become obsolete, are at present in better condition than some other structures of more

recent date, raises the question as to whether there has been much real improvement in the durability of the product in the last forty years.

From all the evidence at hand, there appears to be no question as to the value of the addition of proper silicious materials in the correct proportions, to Portland cement which is to be exposed to the attack of sulphate-bearing waters.

The "Alca" cement has not had sufficient use as yet to enable a positive conclusion as to its value for replacing Portland cement under proper conditions, but the greater part of the evidence indicates that it will probably be a success.

There seems to be no more reason why different cements should not be used for different conditions of exposure than that different timbers or different steels should not be specified to fit the conditions under which they are to be used. It also seems probable that the development of cements that will resist sulphate attack will result in decreasing the cost of structures just as nickel steel used in tension members of large bridges has reduced the cost of those structures. The use of such cements may also make unnecessary some of the precautions now required in the application of Portland cement in structures which must withstand the attack of sulphate-bearing water.

In view of the importance of this subject, as indicated by these discussions and by personal requests for further information, it is hoped that the proposed investigation may have the enthusiastic support of the profession.

IMPROVED TYPE OF MULTIPLE-ARCH DAMS

Discussion*

By Messrs. R. McC. Beanfield, Laurence J. Waller, Evan T. Fisher, and H. E. Linden.

R. McC. Beanfield,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer has read with interest Mr. Noetzli's recent contributions to the science of dam engineering. It appears from these and other publications on the subject that multiple-arch dams can be built economically and with a large factor of safety to much greater heights than was formerly considered practical. The recent completion of the 240-ft. multiple-arch dam on the River Tirso, in Italy, demonstrates the possibilities of this type of dam.

The H-column feature of the improved buttresses is of great importance for high dams, as it provides a safe and economical structure. Such double-wall buttresses are safer than the ordinary type. The economy of such dams is well proved by the fact that they contain only about one-fourth the quantity of concrete of the gravity type of dam. In order to obtain the full economic benefits of the recent improvements in multiple-arch dam design, the possibilities of using large size aggregate in the concrete should be studied.

For high structures, the arches and buttresses are comparatively thick, averaging from 4 to 6 ft. in the lower parts of the dam, in which the use of large size aggregate would be practical.

In the construction of the Don Pedro Dam and of several other dams, in California, the use of aggregate to a size of about 12 in. was economical. For multiple-arch dams, a schedule of maximum sizes of aggregate is suggested in Table 2.

TABLE 2.

Thickness of wall, in feet.	Maximum dimension of aggregate, in inches.
1½ to 3	4
3 to 5	6
5 and greater	10

It is believed that the use of large size aggregate offers the following advantages:

1—Economy in the use of cement;

2.—Economy in crushing and screening operations;

3.—Greater shearing and frictional resistance values; and,

4.—Less surface area, greater density and weight, smaller coefficient of expansion, and less shrinkage of concrete.

* Discussion on the paper by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., continued from December, 1923, *Proceedings*.

† Structural Engr. and Mgr. of Constr., Los Angeles, Calif.

‡ Received by the Secretary, November 15, 1923.

Large cobbles, spalls, or rock, mixed and placed together with concrete in comparatively thick walls, do not impair the strength of the concrete and increase the volume, density, and specific gravity.

Any tendency to shear or slide on planes through the buttresses would be resisted by the large aggregate intersected by such planes. As rock has a shearing strength much greater than concrete, the shearing resistance of the structure would be increased.

The Italian engineers used hand-placed masonry for the buttresses of the Tirso Dam. It is believed, therefore, that concrete containing large size aggregate would combine some of the advantages of hand-laid masonry with the ease and economy of placing concrete. The cost of equipment used for mixing and placing large size aggregate in the Don Pedro Dam was no higher than that for a standard concreting plant.

An efficient plant layout is another vital feature in the economical construction of a multiple-arch dam, and particular study should be given to this phase of the problem. Relative to a construction plant for using large size aggregate, the reader is referred to the description of the methods used on the Don Pedro Dam.*

The writer believes that a shifting trestle system between buttresses and hopper dump cars with a capacity of 2 cu. yd., hauled by gasolene locomotives, will generally be more economical. The concrete can be distributed directly into place from the trestle with comparatively short chute sections. This type of construction plant does not involve hoisting, high towers, and long lines of chutes with the attending difficulties and delays in shifting operations which are numerous in the construction of this class of dam.

In Mr. Noetzli's improved type of buttresses, a cantilever panel form which was satisfactory at the Don Pedro Dam, could be used to advantage. These panel forms could be handled easily in 5-ft. depths and re-used at least ten times.

The construction of a large dam which really becomes a problem of transporting the materials from one place to another, is very intricate and vitally affects the costs. Therefore, more attention and study should be given to the construction plant layout.

LAURENCE J. WALLER,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The multiple-arch type of dam as described in the paper is well adapted to rapid quantity analysis. In this connection, a convenient method is shown in Fig. 27 which gives directly the volume of arches and buttresses for a 60-ft. length of dam comprising one arch and one buttress. The curves have been plotted from estimates on structures similar to the Horseshoe Dam, up to 260 ft. in height.

The quantity determination is as follows: First, plot the profile of the dam, and locate the buttresses, placing them 60 ft. on center. Measure the

* "Some Novel Construction Features on the Don Pedro Dam", *Engineering News-Record*, June 1, 1922.

† Los Angeles, Calif.

‡ Received by the Secretary, November 13, 1923.

height of the buttresses to scale, including the probable depth of excavation. Then, using this height as argument, determine the yardages in each buttress and arch from the corresponding curves. A summation for the various buttresses and arches will give a close quantity estimate for the entire dam.

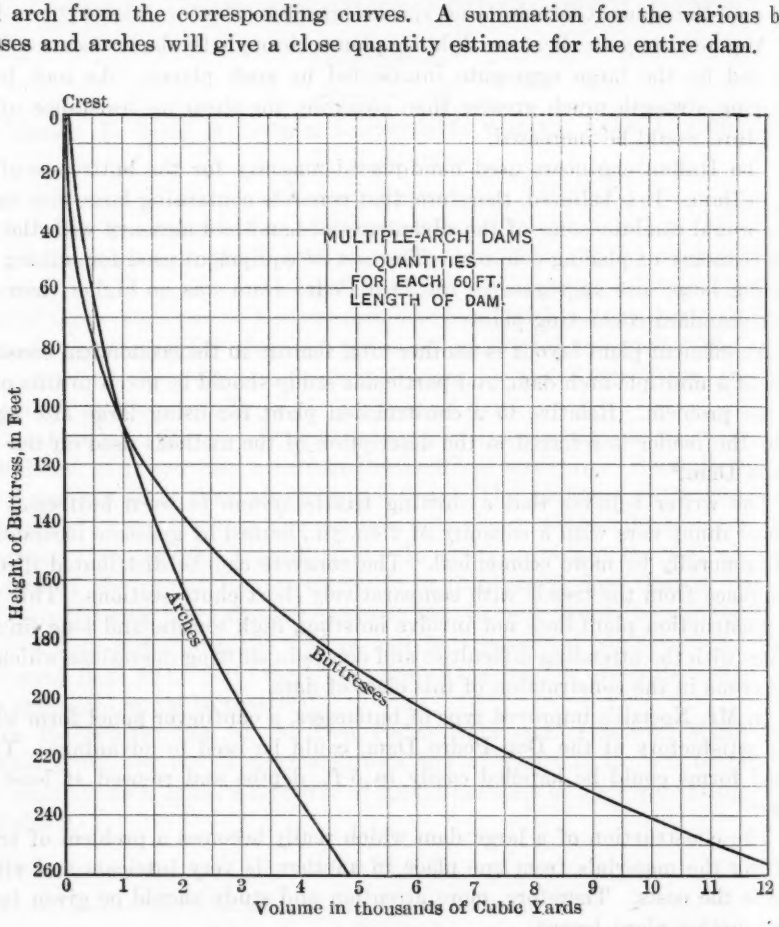

FIG. 27.

EVAN T. FISHER,* ESQ. (by letter).†—The writer is deeply interested in this new type of multiple-arch dam, and has analyzed the stresses in the buttress of the Horseshoe Dam, the results being shown on Fig. 28. The calculations were made according to Mr. Noetzli's monolith or cylinder hoof method, and the results seem to verify his claims for this type of buttress.

All the writer's calculations are undoubtedly a little higher than those actually obtaining, as the buttress webs and the area of the small arch were disregarded except in determining the weight of the structure. As Mr. Noetzli has apparently done, the writer has also assumed the arch to be a semi-circle instead of an ellipse, in order to simplify the calculations.

* San Francisco, Calif.

† Received by the Secretary, November 19, 1923.

It is interesting to note that if the buttress is sloped on a 2:10 batter from the 1 950-ft. level, instead of 1:10 as shown by the author, the maximum stress is decreased from 468 to 400 lb. per sq. in. This adds a very small quantity of concrete, and it would seem that with very little increase in cost, the stress in the buttress could be reduced to 375 or 350 lb. per sq. in., if expedient.

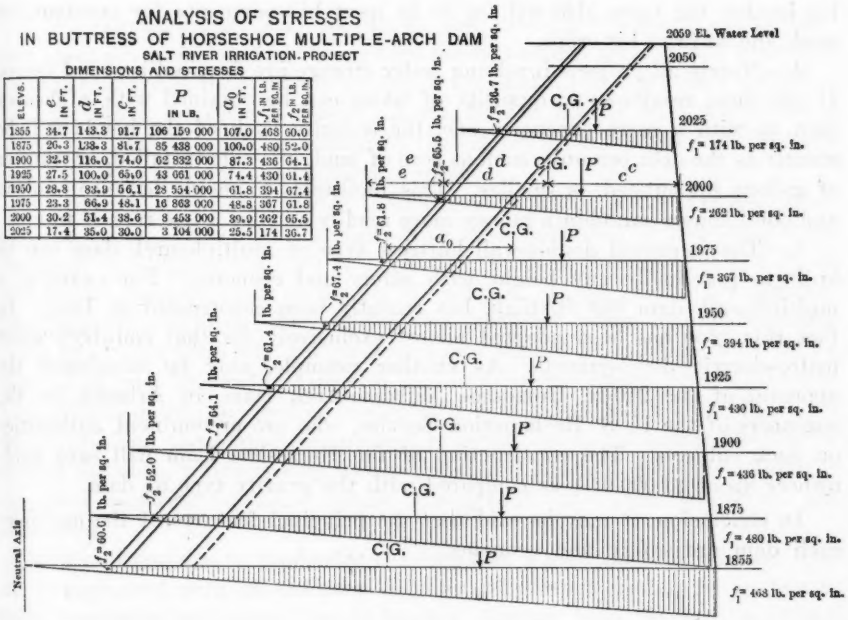

FIG. 28.

The writer agrees with Mr. Noetzli that all fear of spalling of the concrete on the down-stream faces of the arches, due to seepage and subsequent freezing, is unfounded. The New Bear Valley Dam, built in 1910, shows no signs of such action, although at an altitude of 6 500 ft. it is subjected to great changes of temperature in winter. The daily change is often as great as 45 or 50° Fahr. in 24 hours, extending from below zero at night to 40 to 50° at noon.

H. E. LINDEN,* ESQ. (by letter)†.—In the writer's opinion, improvement in dam design, making possible more economical structures, is one of the most important developments in engineering to-day. In this respect, the multiple-arch dam as described by Mr. Noetzli is a great step forward.

The reasons for such a statement may be summarized as follows:

1.—Generally speaking, a multiple-arch dam in a locality suited to this type will cost only half as much as a gravity dam and, in addition, will be twice as safe.

* Cons. Engr., San Francisco, Calif.

† Received by the Secretary, November 30, 1923.

2.—Projects formerly considered too expensive on account of high cost of storage may be feasible if multiple-arch dams are built.

3.—One of the vital factors in land development in the Far West is irrigation. In the majority of cases, it is not possible to acquire a safe and adequate water supply for the various irrigation projects without storage facilities. Such storage facilities have to be paid for by the land owner, and the smaller his burden the more able will he be to meet his payments for construction work and to raise his crops.

4.—Nearly all projects involving water storage are financed by bond issues. If the same results as to quantity of water can be obtained with a cheaper dam as with a more expensive one, the security back of the bonds will be greater as the debt per unit, such as acre of land, kilowatt installed, or number of gallons impounded, is smaller. This facilitates financing because bankers and bond houses can obtain money more readily and for a less rate of interest.

5.—The improved double-wall buttress type of multiple-arch dam can be built to practically any height with safety and economy. For example, a multiple-arch dam 240 ft. high, has recently been constructed in Italy. In fact this type has been adopted almost exclusively for that country's great hydro-electric developments. As another example, may be mentioned the approval of the 210-ft. Horseshoe Multiple-Arch Dam, in Arizona, by the engineers of the U. S. Reclamation Service, who are pre-eminent authorities on such subjects. The construction of the Horseshoe Dam will save landowners about $1 000 000, as compared with the gravity type of dam.

In conclusion, it may be said that the principal features of the multiple-arch dam are safety and economy.

STRESSES IN MULTIPLE-ARCH DAMS

Discussion*

By B. F. JAKOBSEN, M. AM. SOC. C. E.†

B. F. JAKOBSEN,‡ M. AM. SOC. C. E. (by letter).§—Buckling of the buttresses was not considered in the writer's paper, because approximate computations indicated that there was no danger of buckling in well designed multiple-arch buttresses. As Fred A. Noetzli, Assoc. M. Am. Soc. C. E., has proposed a hollow buttress‖ in order to eliminate the danger from buckling, it becomes necessary, therefore, to investigate that subject.

It will be demonstrated that if the buttress shown on Plate III¶ was made solid instead of hollow with the same quantity of material, without steel reinforcement or counterforts (200 ft. high, 3 ft. thick at the top, and 11.2 ft. at the bottom), it would be in no danger of buckling. If steel and a few counterforts properly reinforced were added, the factor of safety against buckling would be greater than 40. As great accuracy cannot be attained, a factor of safety of at least 20 should be insisted on, but even that factor would still make it possible to double the load on the column with safety against buckling. In the calculations, the column is supposed to be cut out of the buttress and to be fixed at the ends, which is very nearly the case. The fact that this column is supported sidewise to some extent throughout its entire length is not considered.

It is well perhaps to emphasize the fact that a multiple-arch buttress cannot be compared with an ordinary building column which may be subject to heavy overloads, excessive eccentric loading, damage from fire, etc., and must be dimensioned accordingly. The buttress receives its greatest load from the water pressure, which is very accurately centered in the axis of the column.

Consider, first, an ordinary column of length, L, and a constant section, with a least radius of gyration, i. For this column, the Mörsch-Ritter equation applies. This equation, which was developed from a consideration of the stress-strain curve of concrete** and from tests on concrete columns made by Professor C. von Bach, is as follows:

$$s_c = \frac{1.25\, s}{1 + 0.0001 \left(\frac{L}{i}\right)^2} \qquad (50)$$

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* Discussion of the paper by B. F. Jakobsen, M. Am. Soc. C. E., continued from December, 1923, *Proceedings*.

† Author's closure.

‡ Hydr. and Elec. Engr., San Francisco, Calif.

§ Received by the Secretary, November 14, 1923.

‖ "Improved Type of Multiple-Arch Dams", *Proceedings*, Am. Soc. C. E., August, 1923, p. 1065.

¶ *Loc. cit.*, p. 1083.

** "Taschenbuch für Bauingenieure", by Max Foerster, Third Edition, 1920, Article 35, p. 194, and also pp. 272, 337, and 338.

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in which,

s_c = permissible stress for column; and

s = permissible stress for the concrete in compression.

From the Euler formula, the permissible stress, with a factor of safety of 10, is:

$$s_c' = \frac{E}{\left(\frac{L}{i}\right)^2} \quad \ldots\ldots (51)$$

The German building code for 1916 prescribes that, if a concrete column is fifteen times longer than the smallest dimension of the section, it must be computed by the Euler formula, using a factor of safety of 10 and $E = 2\,000\,000$ lb. per sq. in.

Using this value in Equation (51) and a value of $s = 500$ lb. per sq. in. in Equation (50), and equating:

$$\frac{L}{i} = 69 \quad \ldots\ldots (52)$$

For values of $\frac{L}{i}$ greater than 69, Equation (51), that is, the Euler formula, will give the smaller value for the permissible loading and, therefore, should be used. For values of $\frac{L}{i}$ less than 69, Equation (50) is the more conservative. This has been verified by actual tests which show that, for large values of $\frac{L}{i}$, the Euler formula is correct, whereas, for smaller values, it is not.*

In order to investigate the buttress for buckling, consider a strip 1 ft. wide cut out of the buttress, the 1 ft. to be measured at right angles to the length of the buttress. The column has a total length of L ft., a variable width of b ft., a sectional area of $b \times 1$ sq. ft., and is loaded with central forces, $P_1, P_2, P_3, P_4, \ldots$, the points of application of which are $x_1, x_2, x_3, x_4, \ldots$, ft. from the bottom, as shown in Fig. 46. The column is assumed to be fixed at both ends.

The column may be dealt with by Vianello's graphical method,† which consists in assuming a deflection line for the column and then computing the moments at a sufficient number of points. The deflections which these bending moments would produce are then determined graphically. In this determination, it should be kept in mind that if the forces, P, are deflected away from the axis, a couple is produced with the reaction at the bottom. In order to counterbalance this couple, a reaction, acting at right angles to the column, must be introduced at the ends. (See Fig. 50.) By trial the two deflection curves are made roughly similar, and the factor of safety is found as the ratio of the two areas formed by the two deflection lines. Ritz's method of approximation‡

* "Materials of Construction", by J. B. Johnson, Fourth Edition, 1912, Chapter XVI, Figs. 296, 297, and 298.

† *Zeitschrift* des Vereins deutscher Ingenieure, 1898, p. 1436.

‡ *Crelles Journal für reine und angewandte Mathematik*, Bd. 135, Heft 1 (1908); also, "Technische Elastizitätslehre", by H. Lorenz, München und Berlin, 1913, p. 397.

is more convenient and consists in assuming an equation for the deflection line, such, for example, as:

$$y = a f(x) + b F(x) + c \phi(x) + \ldots\ldots\ldots\ldots\ldots\ldots (53)$$

in which f, F, ϕ, . . ., are known functions and a, b, c, . . ., are constants, which are to be determined. From the known external forces and Equation (53), the work performed by the external forces, or the external work, W_e, can be calculated. Likewise, the work of deformation or the internal work, W_i, can be calculated, as this depends on the deformations and the known dimensions of the beam or column under consideration. In these two equations, the constants a, b, and c, are the only unknowns. It may be shown that these constants can be determined by partial differentiation of $2 W_e - W_i$ with respect to the constants and equating to zero, that is,

$$\frac{\delta (2 W_e - W_i)}{\delta a} = 0\,; \quad \frac{\delta (2 W_e - W_i)}{\delta b} = 0\,; \quad \frac{\delta (2 W_e - W_i)}{\delta c} = 0 \ldots (54)$$

In this manner, as many equations are obtained as there are constants. The values found for the constants will furnish the best possible solution compatible with Equation (53), that is, compatible with the functions and number of constants assumed.

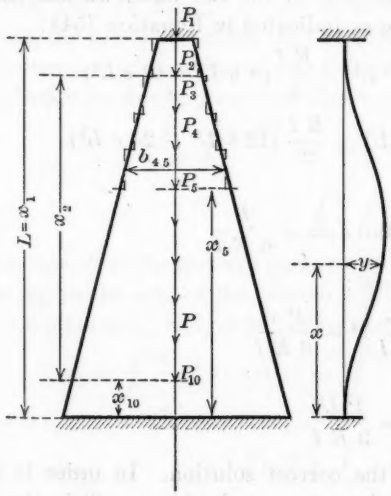

FIG. 46.

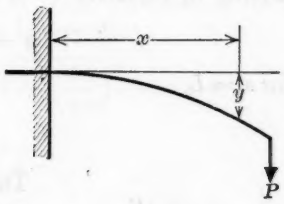

FIG. 47.

As an illustration, assume a beam to be loaded as shown in Fig. 47, and assume that:

$$y = a x + b x^2 + c x^3 \ldots\ldots\ldots\ldots\ldots\ldots (55)$$

Then,

$$\frac{d y}{d x} = a + 2 b x + 3 c x^2 \ldots\ldots\ldots\ldots\ldots\ldots (56)$$

and,

$$\frac{d^2 y}{d x^2} = 2 b + 6 c x \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

For $x = 0$, $\frac{d y}{d x} = 0$ and, therefore, from Equation (56), $a = 0$.

The deflection at the end is:

$$y_m = b\,L^2 + c\,L^3$$

Twice the work done by P, therefore, is:

$$2\,W_e = P\,y_m = P\,(b\,L^2 + c\,L^3) \quad \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

The internal work is:

$$W_i = \int_0^L \frac{M^2\,d\,x}{2\,E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (59)$$

M, expressed as a function of the deformation, neglecting shear, is:

$$M = \frac{E\,I\,d^2\,y}{d\,x^2}$$

Introducing the value of $\frac{d^2\,y}{d\,x^2}$ from Equation (57):

$$M = E\,I\,(2\,b + 6\,c\,x) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (60)$$

Substituting Equation (60) in Equation (59), and integrating:

$$W_i = \frac{1}{2}\,E\,I\,(4\,b^2\,L + 12\,b\,c\,L^2 + 12\,c^2\,L^3) \quad \ldots\ldots\ldots\ldots (61)$$

W_e and W_i are thus expressed as functions of the two unknown constants, b and c, and may be found by proceeding as indicated in Equation (54):

$$\frac{\delta\,(2\,W_e - W_i)}{\delta\,b} = 0 = P\,L^2 - \frac{E\,I}{2}\,(8\,b\,L + 12\,c\,L^2)$$

$$\frac{\delta\,(2\,W_e - W_i)}{\delta\,c} = 0 = P\,L^3 - \frac{E\,I}{2}\,(12\,b\,L^2 + 24\,c\,L^3)$$

Solving:

$$b = \frac{P\,L}{2\,E\,I}, \text{ and } c = +\frac{P}{6\,E\,I}$$

Introducing in Equation (54), as $a = 0$:

$$y = \frac{P\,L}{2\,E\,I}\,x^2 - \frac{P\,x^3}{6\,E\,I}$$

and, for $x = L$,

$$y_m = \frac{P\,L^3}{3\,E\,I}$$

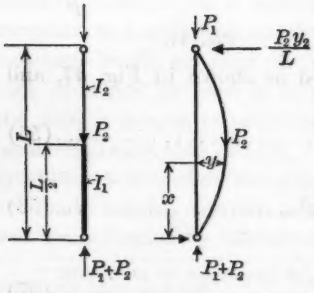

FIG. 48.

This is the correct solution. In order to make certain that the result is a sufficiently close approximation, it would be necessary, in general, to add one or two more members to the expression for y:

$$y = b\,x^2 + c\,x^3 + e\,x^4 + g\,x^5$$

and repeat the calculations. In this case, $e = g = 0$ would result.

Professor A. Ostenfeld* has treated a column, as shown in Fig. 48, both by Engesser's analytical method which is virtually the same as Vianello's

* "Teknisk Elasticitetslære", Third Edition, Copenhagen, 1916, pp. 440 and 442.

graphical method except that, in simple cases, computations instead of a graphical analysis are used, and by Ritz's method, in which:

$$y = a \sin \frac{\pi}{L} x + b \sin \frac{2\pi}{L} x \ldots\ldots\ldots\ldots\ldots\ldots (62)$$

and,

$$y = a \sin \frac{\pi}{L} x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (63)$$

For $I_1 = 2\,I_2 = 2$ and $P_2 = P_1 = 1$, he finds the factor of safety to be:

By Engesser's method, $n = 8.73 \dfrac{E}{L^2}$

By Ritz's method and Equation (62), $n = 9.3 \dfrac{E}{L^2}$

By Ritz's method and Equation (63), $n = 9.87 \dfrac{E}{L^2}$.

It does not make much difference, therefore, in the value found for n, whether Equation (62) or Equation (63) is used, but if a number of forces, P, are involved, the use of Equation (62) greatly complicates the calculations.

The method of procedure is as follows, referring to Fig. 46 and neglecting the deformation due to shear and direct compression. The internal work is:

$$W_i = \frac{1}{2\,E} \int_{x_2}^{x_1} \frac{M^2}{I}\, d\,x = \frac{1}{2\,E} \int_{x_2}^{x_1} \frac{1}{I} \left(E\,I \frac{d^2\,y}{d\,x^2} \right)^2 d\,x$$

$$= \frac{E}{2} \int_{x_2}^{x_1} I \left(\frac{d^2\,y}{d\,x^2} \right)^2 d\,x \ldots\ldots\ldots\ldots\ldots\ldots (64)$$

In this case, I, is the average moment of inertia between x_1 and x_2. The force, P_1, acting in the axis of the column a distance, x_1, from the base, moves downward a distance, u_1, and, as the deflection curve is very flat:

$$u_1 = \int_0^{x_1} (d\,s - d\,x) = \int_0^{x_1} \left(\sqrt{1 + \left(\frac{d\,y}{d\,x} \right)^2} - 1 \right) d\,x,$$

or nearly,

$$u_1 = \frac{1}{2} \int_0^{x_1} \left(\frac{d\,y}{d\,x} \right)^2 d\,x$$

The contribution of P_1 to the external work is, therefore, $W_e = \dfrac{1}{2} P_1\, u_1$, and, consequently:

$$\left.\begin{aligned} 2\,W_e - W_i = {} & \frac{1}{2} P_1 \int_0^{x_1} \left(\frac{d\,y}{d\,x} \right)^2 d\,x + \frac{1}{2} P_2 \int_0^{x_2} \left(\frac{d\,y}{d\,x} \right)^2 d\,x + \ldots. \\ & - \frac{1}{2} E \int_{x_2}^{x_1} I_1 \left(\frac{d^2\,y}{d\,x^2} \right)^2 d\,x - \frac{1}{2} E \int_{x_3}^{x_2} I_2 \left(\frac{d^2\,y}{d\,x^2} \right)^2 d\,x - \ldots. \end{aligned} \right\} (65)$$

Assume:

$$y = a \left(1 - \cos 2\pi \frac{x}{L} \right)$$

Then,

$$\frac{d\,y}{d\,x} = \frac{2\pi}{L}\, a \sin \left(2\pi \frac{x}{L} \right)$$

and,

$$\frac{d^2 y}{d x^2} = \frac{4 \pi^2}{L^2} a \cos\left(2 \pi \frac{x}{L}\right)$$

For $x = 0$ and $x = L$, $y = 0$; for $x = 0$, $x = \frac{L}{2}$ and $x = L$, $\frac{d y}{d x} = 0$, as it should. (See Fig. 46.) Introducing these values in Equation 65, and integrating:

$$\left.\begin{aligned} 2 W_e - W_i = {} & \frac{P_1 2 \pi}{2 L} a^2 \left[\frac{\pi}{L} x_1 - \frac{1}{4} \sin\left(\frac{4 \pi}{L} x_1\right)\right] \\ & + \frac{P_2 2 \pi}{2 L} a^2 \left[\frac{\pi}{L} x_2 - \frac{1}{4} \sin\left(\frac{4 \pi}{L} x_2\right)\right] + \ldots\ldots \\ & - \frac{E I_1}{2} \frac{8 \pi}{L^3} a^2 \left[\frac{x_1 - x_2}{L} + \frac{1}{4}\left(\sin\left(4 \pi \frac{x_1}{L}\right) - \sin\left(4 \pi \frac{x_2}{L}\right)\right)\right] \\ & - \frac{E I_2}{2} \frac{8 \pi}{L^3} a^2 \left[\frac{x_2 - x_3}{L} + \frac{1}{4}\left(\sin\left(4 \pi \frac{x_2}{L}\right) - \sin\left(4 \frac{x_3}{L}\right)\right)\right] - \ldots \end{aligned}\right\} \quad (66)$$

Referring to Fig. 49, $A\ D\ B$ is the axis of the unloaded column and $A\ C\ D$ the axis of the loaded column. The length of the column is not changed, because the influence of direct compression is assumed to be negligible. The equation then becomes:

FIG. 49.

$$L = A\ B = \int_0^{x = A\ D} d\ s = \int_0^{x = A\ D} \sqrt{d\ x^2 + d\ y^2}$$
$$= \text{nearly} \int_0^{x = A\ D} \left(1 + \frac{1}{2}\left(\frac{d\ y}{d\ x}\right)^2\right) d\ x$$

as $\frac{d\ y}{d\ x}$ is always small in comparison with unity.

It is also:

$$L = \int_0^{x = A\ B} d\ x\ ; \text{ and } A\ B = A\ D + \int_0^{x = A\ D} \frac{1}{2}\left(\frac{d\ y}{d\ x}\right)^2 d\ x$$

and, therefore,

$$u_1 = A\ B - A\ D = \int_0^{x_1 - u_1} \frac{1}{2}\left(\frac{d\ y}{d\ x}\right)^2 d\ x$$

as $A\ D = x_1 - u_1$. Introducing the value for $\frac{d\ y}{d\ x}$ given in the text, and integrating:

$$u_1 = \frac{\pi}{L} a^2 \left[\frac{\pi}{L}\left(x_1 - u_1\right) - \frac{1}{4} \sin\left(4 \pi \frac{x_1 - u_1}{L}\right)\right]$$

However, u_1 is small compared to x_1 and L and, therefore, u_1 is changed only slightly if in the integral for u_1 previously given, the upper limit is x_1 instead of $x_1 - u_1$.

The forces designated in Equation (66) as P_1, P_2, ..., are not the actual forces, but those which produce the deflection. If n is the factor of safety and the actual forces are $P_1, P_2, \ldots$, the forces in Equation (66) should be written, $n P_1$, $n P_2$, ... Differentiating Equation (66) with respect to

a, and equating to zero, bearing in mind that a may have any value, but is not zero, the following equation is obtained for the factor of safety:

$$n = 4\,\frac{\pi^2}{L^2}\,E\,\frac{I_1 B_1 + I_2 B_2 + I_3 B_3 + \ldots\ldots}{P_1 A_1 + P_2 A_2 + P_3 A_3 + \ldots\ldots} \qquad (67)$$

and,

$$A_1 = \pi\,\frac{x_1}{L} - \frac{1}{4}\sin\left(4\,\pi\,\frac{x_1}{L}\right)$$

$$A_2 = \pi\,\frac{x_2}{L} - \frac{1}{4}\sin\left(4\,\pi\,\frac{x_2}{L}\right)$$

$$A_3 = \pi\,\frac{x_3}{L} - \frac{1}{4}\sin\left(4\,\pi\,\frac{x_3}{L}\right), \text{ etc.}$$

$$B_1 = \frac{x_1 - x_2}{L}\,\pi + \frac{1}{4}\left[\sin\left(4\,\pi\,\frac{x_1}{L}\right) - \sin\left(4\,\pi\,\frac{x_2}{L}\right)\right]$$

$$B_2 = \frac{x_2 - x_3}{L}\,\pi + \frac{1}{4}\left[\sin\left(4\,\pi\,\frac{x_2}{L}\right) - \sin\left(4\,\pi\,\frac{x_3}{L}\right)\right]$$

$$B_3 = \frac{x_3 - x_4}{L}\,\pi + \frac{1}{4}\left[\sin\left(4\,\pi\,\frac{x_3}{L}\right) - \sin\left(4\,\pi\,\frac{x_4}{L}\right)\right], \text{ etc.}$$

The law is apparent and the coefficients may be formed by proceeding as indicated. If all the forces except P_1 are zero, $I_1 = I_2 = I_3, \ldots$, and $x_1 = L$, the Euler formula results from Equation (67). If there are ten forces equally spaced, P_1 acting at the end and P_{10} acting $x_{10} = \frac{L}{10}$ from the base, the coefficients are those given in Table 3. In order to find the factor of safety, n, from Equation (67), it is only necessary to determine the ten forces, P_1, P_2, P_3, ..., spaced equally and the ten moments of inertia, I_1, I_2, I_3, ..., and then multiply by the coefficients from Table 3, as indicated by Equation (67), and evaluate this equation.

TABLE 3.

$A_1 = 3.14$	$B_1 = 0.552$
$A_2 = 3.068$	$B_2 = 0.223$
$A_3 = 2.663$	$B_3 = 0.020$
$A_4 = 2.053$	$B_4 = 0.223$
$A_5 = 1.648$	$B_5 = 0.552$
$A_6 = 1.57$	$B_6 = 0.552$
$A_7 = 1.496$	$B_7 = 0.223$
$A_8 = 1.089$	$B_8 = 0.020$
$A_9 = 0.481$	$B_9 = 0.223$
$A_{10} = 0.076$	$B_{10} = 0.552$

The recurrence of the values for the coefficients, B, is, of course, due to the fact that a symmetrical deflection line has been assumed. In the equation for y, if two members are used so as to take account of the unsymmetrical deflection, the coefficients, B, would not be exactly alike. It has already been stated that the use of two members in the equation for y, would increase the labor very greatly and that only an approximation can be obtained in any case.

It must be insisted, therefore, that n be so large as to leave no doubt regarding the safety of the structure.

It should be noted that the use of Equation (67) implies that the unit compression due to $P_1 + P_2 + P_3 + \ldots$, does not exceed the permissible compression for the material at any section, say, 500 to 600 lb. per sq. in. for concrete. Equation (67) is derived on the assumption that failure would occur from buckling if the load is sufficiently increased, which implies that failure in compression is not possible. If this is not true, Equation (67) is not true. If, for example, in Equation (67), one of the forces, P, is greatly increased, it might change n very little (especially if the force is near the bottom, where the coefficient, A, is small), but the direct compression at that section would soon exceed the permissible value and Equation (67) cannot be applied.

In order to apply Equation (67) to the buttress shown in Plate III of Mr. Noetzli's paper, previously mentioned (the buttress being solid, but having the same cross-sectional area as the hollow buttress), it will be assumed that the unit compression varies uniformly from 500 lb. per sq. in. (Mr. Noetzli gives 485 lb. per sq. in. in his Fig. 10*) at the base to zero at the top. The water load midway is only one-quarter that at the base, and the same is approximately true for the stress due to the weight of the structure, so that the loading is believed to be a fairly correct assumption. As a check, the stress in Point B of the writer's Fig. 33† is, $\sigma_x = 319$ lb. per sq. in. (see Table 1‡), and at Point D it is $112.5 + 80.4 = 192.9$, that is 60% of the stress at Point B (see, also, Table 1). Point B is 100 ft., and Point D is 70 ft. below the top. When designing a dam, of course, the actual loads should be used.

The buttress varies from 3 ft. at the top to 11.2 ft. at the bottom. P_1 acts $x_1 = 200$ ft. from the base ($L = 200$ ft.); and the interval between forces is $\frac{L}{10} = 20$ ft. P_1 is found by determining the width of the buttress 190 ft. from the base, and multiplying this figure by the unit stress at that point. I_1 is found from the width 190 ft. from the base, I_2 from the width 170 ft. from the base, etc. P_2 is found in the same manner as P_1, by considering the dimensions and unit compression 170 ft. from the base and subtracting P_1. The results are shown in Table 4. Introducing these values in Equation (67), gives:

$$n = 4\,\frac{\pi^2}{200^2}\,2\,000\,000 \times 144\,\frac{141.6}{1\,025\,300} = 39.4$$

This computation does not take into account the reinforcing steel or the counterforts, but it shows that there is no danger of buckling and that there is no reason for using hollow buttresses.

As stated previously, the Euler formula applies only to slender columns. In this case, the least radius of gyration at the top is, $i = 0.865$, and, at the middle, $i = 2.04$, so that the ratio $\frac{L}{i}$ for the top and middle, respectively, is 232 and 100.

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* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1088.
† *Loc. cit.*, p. 1116.
‡ *Loc. cit.*, p. 1117.

TABLE 4.

$x_1 = 200$ ft.	$P_1 = 12\,300$ lb.	$b_{1,2} = 3.41$ ft.	$I_1 = 3.3$ ft.^{4}
$x_2 = 180$ "	$P_2 = 33\,300$ "	$b_{2,3} = 4.23$ "	$I_2 = 6.3$ "
$x_3 = 160$ "	$P_3 = 45\,300$ "	$b_{3,4} = 5.05$ "	$I_3 = 10.7$ "
$x_4 = 140$ "	$P_4 = 57\,100$ "	$b_{4,5} = 5.87$ "	$I_4 = 16.9$ "
$x_5 = 120$ "	$P_5 = 69\,000$ "	$b_{5,6} = 6.69$ "	$I_5 = 25.0$ "
$x_6 = 100$ "	$P_6 = 81\,000$ "	$b_{6,7} = 7.51$ "	$I_6 = 35.3$ "
$x_7 = 80$ "	$P_7 = 93\,000$ "	$b_{7,8} = 8.33$ "	$I_7 = 48.1$ "
$x_8 = 60$ "	$P_8 = 105\,000$ "	$b_{8,9} = 9.15$ "	$I_8 = 63.7$ "
$x_9 = 40$ "	$P_9 = 116\,000$ "	$b_{9,10} = 9.97$ "	$I_9 = 82.8$ "
$x_{10} = 20$ "	$P_{10} = 127\,000$ "	$b_{10,11} = 10.79$ "	$I_{10} = 104.6$ "

The limiting value found in Equation (52) was 69, so that Equation (67) or the Euler formula is applicable to this column.

It is interesting to note that if the column is assumed to be hinged at the top and bottom, that is, if Equation (63) is used for y, the equation for n is the same as Equation (67), except that the 4 disappears and the coefficients have the values given in Table 5.

TABLE 5.

$A_1 = 1.57$	$B_1 = 0.01$
$A_2 = 1.268$	$B_2 = 0.066$
$A_3 = 1.02$	$B_3 = 0.157$
$A_4 = 0.862$	$B_4 = 0.248$
$A_5 = 0.796$	$B_5 = 0.304$
$A_6 = 0.786$	$B_6 = 0.304$
$A_7 = 0.776$	$B_7 = 0.248$
$A_8 = 0.709$	$B_8 = 0.157$
$A_9 = 0.552$	$B_9 = 0.066$
$A_{10} = 0.304$	$B_{10} = 0.01$

This gives the value of $n = 7.18$ instead of 39.4, or a ratio of 5.5 instead of 4, which is the ratio for an ordinary column with constant section and only one load applied at the end. The greater ratio in the present case was to be expected, due to the fact that the moment of inertia increases rapidly near the lower end of the column.

Applying the formula to the buttress shown in the writer's Fig. 33 previously mentioned, and assuming hinged ends, gives a value of $n = 13.4$. This computation does not take into account reinforcing steel or counterforts. For the column considered as fixed at the end, n would be about 73.

The writer has given the method in some detail and also the coefficients, A and B, because he believes it to be generally applicable to multiple-arch buttresses and, in general, that it will suffice to consider ten sections. The influence of reinforcing steel and counterforts can be taken into account by dividing the moment of inertia at any section by the length or rather depth of the buttress.

Another method of checking the column in Mr. Noetzli's dam is, as follows: The German building code (1916) prescribes that if the length of a column is more than twenty times the least dimension, the load shall then be

considered as applied $\frac{1}{200}$ off center. Applying this to the column, considered as solid, and with all loads 1 ft. off center, it is found that:

x, in feet.....	=	190	170	150	130	110	90	70	50	30	10
s, in pounds per square inch...	=	81	189	280	354	426	494	558	624	680	740

This regulation, however, is hardly applicable to a column loaded at intervals. A more rational use of this method would be to assume the eccentricity of each force as $\frac{1}{200}$ of its distance from the base, but, nevertheless, a fair indication of the sufficiency of the column in question is obtained.

Whether the buttress may be assumed as fixed at the ends, as has been done in order to derive Equation (67), remains to be investigated. If the buttress is an ideal column, the forces will produce no bending until n approaches unity. Careful tests recently made by Th. von Kàrmàn* show that when the load on a column is about 90% of that which produces failure, the deflection is still exceedingly small. At about this load, however, the deflection increases very rapidly, which is in conformity with theory.

It has already been shown that, if the forces are 1 ft. off center, the maximum compression at the base is only 740 lb. per sq. in., and that as the forces act well within the middle third, the entire base is under compression.

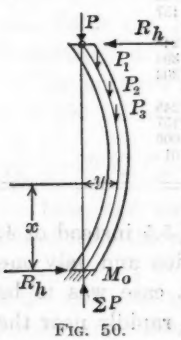

FIG. 50.

The buttress, however, might be built slightly crooked or lack of homogeneity might make it act as a slightly curved column, in which case there would be a bending moment at the base. If tension is produced as a result of this bending moment superimposed on the compression, the buttress cannot be considered as fixed at the base. At the top, it may be considered as fixed, but it will not make much difference whether it is considered fixed or hinged, owing to the small section at the top.

Assume the column to have a shape as shown in Fig. 50, hinged at the top and fixed at the base. The forces, P, produce a couple, $\Sigma P y$, tending to rotate the entire structure clockwise. There must be, therefore, a horizontal reaction, R_h, at the top and at the bottom, and a moment, M_0, at the base.

Therefore,

$$\Sigma (P \times y) = M_0 + R_h \times L \quad \ldots\ldots (68)$$

The stress at the base is:

$$\sigma = \frac{\Sigma P}{b_0} \pm \frac{6 M_0}{b^2_0} \quad \ldots\ldots (69)$$

As a limiting value, $\sigma = 0$, and, therefore,

$$M_0 = \frac{b_0}{6} \Sigma P \quad \ldots\ldots (70)$$

* "Untersuchungen über Knickfestigkeit", in den *Mitteilungen über Forschungsarbeiten*, Vereins deutscher Ingenieure, 1910, Heft 81.

From Equation (68):

$$M_0 < \Sigma P y = y_{av} \Sigma P \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (71)$$

The meaning of y_{av} is evident. It is less than the maximum deflection, the amount depending on the magnitude and location of the forces, P. Combining Equations (70) and (71):

$$y_{av} > \frac{b_0}{6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (72)$$

For the buttress of Mr. Noetzli's dam, $y_{av} > \frac{11.2}{6} = 1.87$ ft. The axis of the column, however, cannot be assumed to be more than 1.87 ft. out of true and, consequently, the column may be considered as fixed at the base, provided the foundation is rock, or that it is, and will, remain as unyielding as a rock foundation. It may be well to consider wind pressure. Mr. Noetzli* uses a pressure of 15 lb. per sq. ft., acting under an angle of 45° to the buttress, which figure corresponds to a wind velocity† of $0.0032\ V^2 = 15$; or $V = 68.5$ miles per hour. This is a quite conservative assumption and only a very small part of the buttress could be effectively exposed to this pressure. Assuming the buttress to be fixed at the base and supported at the top, the moment at the base is approximately,

$$M_0 = \frac{p\,L^2}{8} = \frac{10.6 \quad 200^2}{8} = 53\,000 \text{ ft-lb.}$$

and the maximum bending stress produced is,

$$\sigma = 6\,\frac{M_0}{I} = 6\,\frac{53\,000}{11.2^2 \quad 144} = 15.6 \text{ lb. per sq. in.}$$

The compression at the base with reservoir full is about 500 lb. per sq. in. At the middle point of the buttress, the moment is, approximately,

$$M = \frac{1}{16}\,p\,L^2 = 26\,500 \text{ ft-lb.}$$

and the maximum bending stress is,

$$\sigma = 6\,\frac{26\,500}{7.1^2 \quad 144} = 21.9 \text{ lb. per sq. in.}$$

The compression at the middle point is about 250 lb. per sq. in. for reservoir full. The wind stress of 21.9 lb. per sq. in. cannot produce tension even when the reservoir is empty. The weight of the buttress alone would produce a compression of:

$$\frac{3 + 7.1}{2 \times 7.1}\,100 \times \frac{150}{144} = 75.2 \text{ lb. per sq. in.}$$

It would seem, therefore, that the influence of the wind stresses is negligible.

The comparison just given, between Mr. Noetzli's hollow buttress and a solid one containing the same cross-sectional area, is very unfair to the solid

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1089.

† Test at the Eiffel Tower, Merriman's "American Civil Engineers' Handbook", Fourth Edition, 1920, p. 571.

buttress, as it would cost much less. A fairer comparison could be had by considering a solid buttress having from 30% to, perhaps, 50% more sectional area than the hollow buttress.

In his discussion,* Professor Cain calls attention to the importance of realizing that oblique sections cannot be used for correct stress determinations and derives the writer's Equation (23)† in a different and more direct manner. As oblique sections are often used in practise,‡ the question will be investigated further. A beam, as shown in Fig. 51, is an indeterminate structure and the stress distribution may be determined by the method of least work.§ The writer's Equation (23), previously mentioned, was derived on the assumption that plane oblique sections remain plane. The total internal work of deformation may then be expressed as a function of tan α and that value of tan α determined which gives the least work. Based on the assumption that the oblique plane remains plane in the beam shown in Fig. 51, the stress parallel to the center line from Equation (23) is:

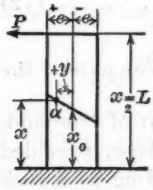

FIG. 51.

$$\sigma_x = \frac{y}{I} P (L - x_0) + \frac{P}{A} \tan \alpha \ldots\ldots\ldots\ldots\ldots\ldots (73)$$

in which I and A are the moment of inertia and the area, respectively, of a normal section.

The total internal work is:

$$W_i = \frac{1}{2} \int_{y=-e}^{y=+e} dy \int_{x=x_1}^{x=x_2} \frac{\sigma_x^2}{E} dx \ldots\ldots\ldots\ldots\ldots\ldots (74)$$

In this equation:

$$x = x_0 + y \tan \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (75)$$

and the limits are:

$$x_1 = 0; \text{ and } x_2 = L$$

Eliminating x_0 from Equation (73) by Equation (75), and introducing into Equation (74), the value thus found for σ_x, then differentiating W_i with respect to tan α, integrating and equating to zero gives tan $\alpha = 0$. This is dependent on the assumption that plane sections remain plane, but it indicates that oblique sections may not be assumed to remain plane. This assumption is limited to a rectangular section of constant width throughout, and the shear is assumed to be negligible.

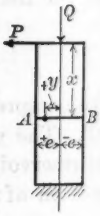

FIG. 52.

It is possible, however, to give a general demonstration. In Fig. 52 assume that the shear deformation is negligible and also neglect the lateral deformations and assume a relation between the stresses and the strains as given by Equation (76). No assumption

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1785.

† *Loc. cit.*, August, 1923, p. 1110.

‡ In wedge-shaped reinforced concrete beams, see "Earth Pressure, Retaining Walls and Bins", p. 239, by William Cain, M. Am. Soc. C. E.; "Concrete, Plain and Reinforced", Second Edition (1912), p. 668, by Messrs. F. W. Taylor and S. E. Thompson; and, also, in determining the stresses in the down-stream toe of gravity dams, see, "Engineering for Masonry Dams", by W. P. Creager, M. Am. Soc. C. E., 1917, p. 72.

§ See, for example, "Determination of the Stresses in Elastic Systems by the Method of Least Work", by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXIV (April, 1891), p. 265.

is made on the distribution of the stress over the section nor whether any plane sections remain plane. The stress-strain relation is:

$$\text{Elongation per unit length} = \frac{(\text{stress})^n}{k} \quad \ldots\ldots\ldots\ldots\ldots (76)$$

in which n and k are constant. If $n = 1$, $k = E$, the ordinary modulus of elasticity. The conditions are:

$$\int_{y=-e}^{y=+e} \sigma_x \, y \, d\, y = M = P\, x; \quad \frac{Q}{2_e} = c \ldots\ldots\ldots\ldots\ldots (77)$$

The elongation per unit length is, therefore,

$$\frac{(\sigma_x + c)^n}{k}$$

and the internal work for a very short part of the beam is:

$$W_i = \frac{1}{2} \int_{y=-e}^{y=+e} \frac{(\sigma_x + c)^{n+1}}{k} \, d\, y \ldots\ldots\ldots\ldots\ldots (78)$$

σ_x as a function of y is now sought such that Equation (78) becomes a minimum. In this equation, σ_x is the normal bending stress in the normal section, A-B. By variational calculus:

$$\delta\, W_i = \delta \left[a \int_{-e}^{+e} \sigma_x \, y \, d\, y + \frac{1}{2} \int_{-e}^{+e} \frac{(\sigma_x + c)^{n+1}}{k} \, d\, y \right] = 0 \ldots (79)$$

As a is a constant, this expression may be written as:

$$\delta\, W_i = \delta \left\{ \int_{-e}^{+e} \left[a\, y\, d\, y\, \delta\, \sigma_x + a\, \sigma_x\, d\, y\, \delta\, y + a\, \sigma_x\, y\, d\, \delta\, y \right] \right.$$

$$\left. + \frac{1}{2\,k} \int_{-e}^{+e} (n+1)\,(\sigma_x + c)^n \, d\, y\, \delta\, \sigma_x + \frac{1}{2\,k} \int_{-e}^{+e} (\sigma_x + c)^{n+1} \, d\, \delta\, y \right\} = 0$$

By partial integration of the third and fifth members and arranging:

$$\delta\, W_i = a\, \sigma_x\, y\, \delta\, y \int_{-e}^{+e} + \frac{(\sigma_x + c)^{n+1}}{2\,k}\, \delta\, y \int_{-e}^{+e} - \int_{-e}^{+e} (a\, y\, d\, \sigma_x$$

$$+ \frac{n+1}{2\,k} (\sigma_x + c)^n \, d\, \sigma_x)\, \delta\, y + \int_{-e}^{+e} a\, y\, d\, y + \frac{n+1}{2\,k} (\sigma_x + c)^n \, d\, y)\, \delta\, \sigma_x = 0$$

As $\delta\, y$ is zero at the limits and $\delta\, y$ and $\delta\sigma_x$ are independent of each other, this equation reduces to:

$$a\, y + \frac{n+1}{2\,k} (\sigma_x + c)^n = 0$$

or,

$$\sigma_x + c = \left(- a\, y \frac{2\,k}{n+1} \right)^{\frac{1}{n}} \ldots\ldots\ldots\ldots\ldots (80)$$

Equation (77) furnishes the value of the constant, a:

$$\int_{-e}^{+e} \sigma_x \, y \, d\, y = 2 \int_{o}^{e} \left[\left(- a \, y \, \frac{2\,k}{n+1} \right)^{\frac{1}{n}} - c \right] y \, d\, y = M$$

$$= 2 \left(- a \, \frac{2\,k}{n+1} \right)^{\frac{1}{n}} \frac{e^{2+\frac{1}{n}}}{2+\frac{1}{n}}$$

Introducing Equation (80):

$$2 \left(\frac{\sigma_x + c}{y^{\frac{1}{n}}} \right) \frac{e^{\left(2+\frac{1}{n}\right)}}{2+\frac{1}{n}} = M$$

If $\sigma_{x\ max.}$ is the maximum bending stress, then, for $y = e$:

$$\sigma_{x\ max.} = \pm \frac{2+\frac{1}{n}}{2} \frac{M}{e^2} - c \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (81)$$

For $n = 1$, this expression reduces to the standard bending formula,

$$\sigma_{x\ max.} = \pm \frac{M}{I} e - c$$

This proves that the ordinary bending formula is correct for normal sections; it is incorrect if applied to oblique sections (see Table 3).

According to tests by Bach,* $n = 1.16$ for 1 : 2.5 : 5 concrete, and, if $c = \sigma_{x\ max.}$, $\sigma_{x\ max.} = 0.717 \frac{M}{e^2}$ whereas, under the same conditions for $n = 1$, $\sigma_{x\ max.} = 0.75 \frac{M}{e^2}$. As the error amounts to less than 5%, it is on the side of safety and is negligible.

Bach† has derived a formula based on Equation (76) and on the further assumption that plane normal sections remain plane, and he finds the same formula as Equation (81), when $c = 0$.

Equation (81) is derived on the assumption that pure bending acts, for example, by a couple at the end of the beam; only then is the shear zero.

In a multiple-arch dam, stresses, σ_x, σ_y, and τ, exist, due to the loading of the dam and also to the variation of the section. For Section II of the writer's Fig. 33, previously mentioned, the stresses for the buttress are given in Table 2‡. The work of deformation for the σ_x, σ_y, and τ stresses for this section (for the buttress only) is in the ratio, 0.415 : 0.325 : 1.1, if the shear modulus is taken as $\frac{E}{2.4}$. It is seen that the internal work due to shear is by far the greatest. If a beam is stressed in shear and the other stresses are negligible, the principle of least work demands that the shear be distributed evenly over

* "Elasticität und Festigkeit", Third Edition, 1898, p. 58.

† *Loc. cit.*, p. 200.

‡ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1123.

the section. This corresponds to the bending stress concentrated at the extreme fiber and, therefore, this stress would be infinite, unless the bending is actually zero. If the shear is zero, the stress varies uniformly with y, that is, $\sigma_x = a\,y$, in which a is a function of x, but not of y. For the case in which shear greatly predominates and determines the stress distribution, $\sigma_x = a\,y^\infty$. The actual stress distribution for a multiple-arch dam will be between these two extremes.

Fig. 53 (a) shows the linear stress distribution of σ_x; Fig. 53 (b) shows by full lines the corresponding stress distribution for the shear, and by dotted lines the shear distribution which corresponds to minimum internal work due to shear. Fig. 53 (c) shows the stress distribution for σ_y in a rectangular section corresponding to the σ_x stress shown in Fig. 53 (a); the dotted line in Fig. 53 (c) shows the stress distribution corresponding to minimum shear. These diagrams apply to a dam of rectangular section, the weight of the struc-

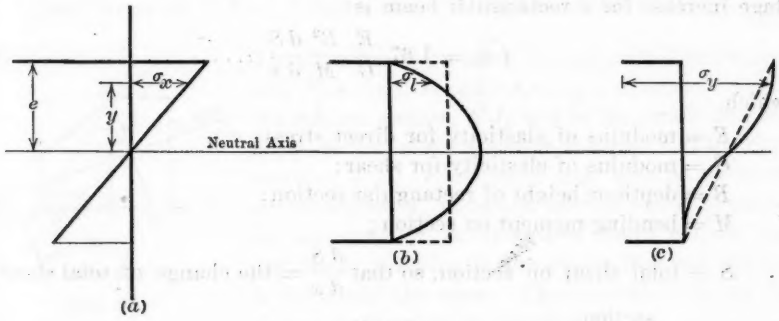

FIG. 53.

ture not being considered. If the weight had been considered, however, only Fig. 53 (a) would have been changed, the other two diagrams, Fig. 53 (b) and Fig. 53 (c) being unaffected. The internal work for shear corresponding to the two conditions of Fig. 53 (b) is shown by the two curves in Fig. 54 (a),

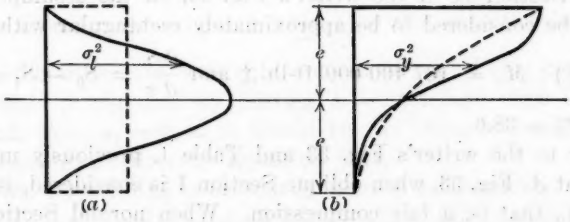

FIG. 54.

and the internal work corresponding to the σ_y stress distribution shown in Fig. 53 (c) is shown in Fig. 54 (b). It should be kept in mind that the stress distribution indicated by the dotted lines in Fig. 53 (b) and Fig. 53 (c) correspond to σ_x infinite at the extreme fiber and zero elsewhere and that these dotted lines, therefore, represent limiting cases which cannot exist in practice. From Fig. 54 (a) and Fig. 54 (b), it was found by the planimeter that although the σ_x stress changes from linear to infinite at the extreme fiber and the work

due to σ_x becomes infinite, the work of deformation for shear is decreased only about 20% and the internal work of the σ_y stress is decreased only about 15 per cent.

If the σ_x stress is assumed to vary less rapidly than y, the work of deformation of all three stresses is increased, which cannot take place. Therefore, wherever shear is present, the σ_x stress must vary faster than y and, consequently, the $\sigma_{x_{max.}}$ stress determined by the ordinary bending formula errs on the side of danger. The writer has not been able to solve the equation for general stress distribution when all three stresses are present in a rectangular beam. For a multiple-arch dam with a complicated variable section the problem becomes very difficult.

J. P. J. Williams,* M. Am. Soc. C. E., has given a formula for determining the increase of the bending stress due to the influence of shear. The percentage increase for a rectangular beam is:

$$i\,\% = 1.67\,\frac{E}{G}\,\frac{B^2}{M}\,\frac{dS}{dx} \qquad (82)$$

in which,

E = modulus of elasticity for direct stress;
G = modulus of elasticity for shear;
B = depth or height of rectangular section;
M = bending moment on section;
S = total shear on section, so that $\frac{dS}{dx}$ = the change of total shear at section.

According to this formula, as Mr. Williams has stated, "where the shear is constant, there is no error whatever in the common theory." This, however, has been shown to be in error; when shear is present, it always operates to increase the maximum bending stress. The Williams formula, however, may be used as the best approximation available at present. Applying it to Section II of the writer's Fig. 33, on the assumption that this section may be considered to be approximately rectangular with $B = 94.5$ ft. (see Fig. 37†): $M_0 = 187\,400\,000$ ft-lb.,‡ and $\frac{dS}{dx} = S_0 - S_1 = 202\,000$ lb.

This gives $i\,\% = 38.6$.

Referring to the writer's Fig. 33 and Table 1, previously mentioned, the stress at Point A, Fig. 33, when oblique Section I is considered, is $\sigma_x = +32.1$ lb. per sq. in., that is, a fair compression. When normal Section II is considered and the influence of shear is taken into account, as determined previously, the stress is: $\sigma_x = 83.2 - 1.386 \times 111.8 = -69.6$ lb. per sq. in. In the first case, the principal stress is 33.4 lb. per sq. in. compression, whereas, when correctly computed, it is 95 lb. per sq. in. tension. This is a considerable difference and cannot be neglected.

* In his discussion of the paper, "Faults in the Theory of Flexure", by Henry S. Prichard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 932.
† *Proceedings*, Am. Soc. C. E., August, 1923, p. 1120.
‡ *Loc cit.*, p. 1122.

It has already been pointed out that the arch may swell. If it is not feasible to prevent swelling by embedding longitudinal steel, the effect on the distribution of stress must be carefully investigated, as it may cause heavy tensile stresses in the buttress.

Professor Cain* has pointed out that, in designing the toe of a retaining wall or a dam, the usual method of considering oblique sections leads to inconsistent results, if the angle, ϕ, in Fig. 55 is fairly large. Such results are not obtained if normal sections are considered. In Fig. 55, it is:

$$z = \frac{x}{\cos \phi} + \frac{h}{2} \sin \phi$$

The bending moment is:

$$M = z\, P \cos \phi = P \left(x + \frac{h}{4} \sin 2\, \phi \right)$$

The normal pressure is $N = P \sin \phi$, and the moment of inertia is $I = \frac{h^3}{\cos^3 \phi}$.

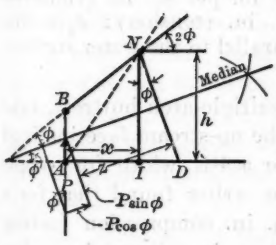

FIG. 55.

The distributed pressure is represented by its resultant, P, acting, as indicated, at a distance x ft. from the oblique section, $N\,I$, and in the center line of the beam, so that $P \sin \phi$ acts in the center of the normal section, $N\,D$. P evidently may also be assumed to act at Point A, in which case $P \sin \phi$ produces a moment and it is, therefore, more convenient to assume P as acting in the center line and the result is obviously the same. The normal stress at N on the normal section is, therefore:

$$\sigma_x = \frac{P}{2\,h} \sin 2\, \phi + 6\, \frac{P}{h^2} \left(x + \frac{h}{4} \sin 2\, \phi \right) \cos^2 \phi$$

and the greatest principal stress is:

$$\sigma_1 = \frac{P}{h} \tan \phi + 6\, \frac{P}{h^2} \left(x + \frac{h}{4} \sin 2\, \phi \right)$$

By the nature of the problem, $\tan 2\, \phi \leq \frac{h}{x}$; if this condition is not fulfilled h and x cannot both remain constants when ϕ varies. For a corner, such as $A\,N\,I$, the stress is obtained by putting $h = x \tan 2\, \phi$. The limit is then $2\,\phi = 90°$, and, then, $\sigma_1 = 0$, as it should be. If there is considerable shear, σ_x must be increased as given by Equation (82).

The equation for bending is ordinarily derived by assuming:

1.—That plane normal sections remain plane and normal to the center line;
2.—That E is constant;
3.—That the shear deformation is negligible; and
4.—That the beam is fixed along a normal plane.

These assumptions have been verified experimentally. It has been shown previously† that when a beam is fixed along an oblique section, no section

* "Earth Pressure, Retaining Walls and Bins", p. 239.

† *Proceedings*, Am. Soc. C. E., August, 1923, p. 1106, and Professor Cain's discussion, *loc. cit.*, October, 1923, p. 1785.

remains plane* and the first assumption is, therefore, not complied with. In view of previous statements, it would seem that the assumption is safe and approximately correct that the bending stress in a normal section varies linearly with y, even for normal sections lying below Section II in the writer's Fig. 33, except for such modification as that brought about by the τ and σ_y stresses. This is true, because the stresses spread gradually into the foundation. A better appreciation of this will be gained from a study of a paper by George Paaswell,† M. Am. Soc. C. E., in conjunction with the marked change in stress distribution in a gravity dam as the base is approached and as shown by Fig. 45‡ and by Professor Cain's discussion.‡ There is considerable information regarding what happens near the foundation and in the foundation itself.

Professor Cain's query§ has been answered by him in a letter to the writer, as follows:

"Considering a buttress 1 ft. thick similar to the one shown in Fig. 33, but 110 ft. high, I found for Section I, $\sigma_2 = +$ 20.1 lb. per sq. in. (compression), but for Section II, $\sigma_2 = -$ 6.48 lb. per sq. in. (tension); σ_2 is the principal stress parallel to the up-stream face, *i. e.*, parallel to the water surface. The results are in line with your computations."

In order to make the load more like that of a multiple-arch buttress, take a buttress 1 ft. thick, 100 ft. high, base, 100 ft., with the up-stream face inclined 50° to the horizontal and with the water pressure for a 4-ft. width (or weight of water $= 4 \times 62.5$). For a horizontal section, the writer found that for a point in the up-stream face, $\sigma_x = 19.2$ lb. per sq. in. compression (acting vertically), and by Equation (31)‖, $\sigma_2 = 2.1$ lb. per sq. in compression. For a normal section at that point, $\sigma_x = 50.5$ lb. per sq. in. tension and $\sigma_2 = 70.1$ lb. per sq. in. tension. This should be augmented as outlined previously, Equation (82), $i =$ about 30%, so that $\sigma_x = 96$ lb. per sq. in. tension and $\sigma_1 = 126$ lb. per sq. in. tension. Instead of a compression of 19.2 lb. per sq. in., as found by the usual method, there is a tension of 126 lb. per sq. in.

The increase in the extreme fiber stress in a gravity dam due to shear deformation is worthy of discussion. The Williams formula is derived for a beam of constant section, and the shear distribution for a rectangular beam would be parabolic. The shear distribution for a gravity dam is linear, zero at the up-stream face and a maximum at the down-stream face.¶ It is not clear to the writer to what extent the Williams formula is applicable to a gravity dam or a multiple-arch dam, but as the error in neglecting the shear deformation is on the side of danger, it is not good engineering to neglect it, especially when considering gravity dams with which even a small tension in the up-stream face across construction joints may be dangerous. Good engineering,

* Professor Cain has informed the writer that the oblique plane section, *C-D*, in Fig. 43, of his discussion previously mentioned, is deformed under stress into a curved surface of slight curvature, concave upward. This was deduced from the equations given in *Proceedings*, Am. Soc. C. E., October, 1923, p. 1786.

† "Transmission of Pressure Through Solids and Soils, and the Related Engineering Phenomena", *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1563.

‡ *Proceedings*, Am. Soc. C. E., October, 1923, pp. 1788–1789.

§ *Loc. cit.*, p. 1789.

‖ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1115.

¶ See *Proceedings*, Am. Soc. C. E., August, 1923, reference to "Stresses in Masonry Dams", p. 1106.

therefore, requires that the compression in the up-stream face, as computed by the ordinary formulas, be made greater than the possible error.

For a horizontal section of a triangular gravity dam the up-stream face of which is vertical and the water level even with the top, the base equal to 65% of the height, and the concrete weighing 150 lb. per cu. ft.: $S = 31.25h^2$, and $M = 10.6\,h^3 - 75 \times b\,h \times \frac{b}{6} = 5.32\,h^3$. With $\frac{E}{G} = 2.4$, the Williams formula gives $i = 20\%$, and the stress in the up-stream face is:

$$75\,h - 1.2 \times 6 \times 5.32 \times \frac{h^3}{b^2} = 15.6 \times h \text{ lb. per sq. ft. tension.}$$

If the shear deformation is neglected, there is a compression of $0.6\,h$ lb. per sq. ft. For $h = 100$, the tension is only 10.8 lb. per sq. in., and for $h = 300$ ft., it is 32.5 lb. per sq. in. Therefore, when designing a high gravity dam, it is not safe to rely too implicitly on the middle-third law which has produced safe structures in the past. This question needs careful consideration.

A part of Mr. Noetzli's discussion,* has been already answered, but the writer does not follow his argument relative to the cracks in the buttress of the Lake Hodges Dam (Fig. 42†). These cracks are perpendicular to the groin line only near the groin, whereas near the foundation they are vertical. Mr. Noetzli's conclusion that multiple-arch dams can be built economically in steps is interesting.

Mr. Jorgensen's‡ point is well taken that if the slope of the arches is too steep, there may be danger of sliding along the construction joints. The allowable stress should be fixed with this in mind and the construction joint should be strengthened, perhaps, by sloping or stepping it and even by reinforcing it. The writer cannot agree with Mr. Jorgensen's remark regarding Professor Cain's formulas§ as applied to the Lake Spaulding Dam. In his discussion‖ of Professor Cain's paper, L. J. Mensch, M. Am. Soc. C. E., pointed out that whenever considerable tension is developed in a concrete arch, a secondary arch free from tension must be considered, and Professor Cain, in his closure,¶ showed that his formulas could be applied also to this case.** If fine hair

* *Proceedings*, Am. Soc. C. E., December, 1923, p. 2069.

† *Loc cit.*, August, 1923, p. 1128.

‡ *Loc. cit.*, November, 1923, p. 1927.

§ "The Circular Arch under Normal Load", *Transactions*, Am. Soc. C. E., Vol. LXXXI (1922), p. 233.

‖ *Loc. cit.*, p. 264.

¶ *Loc. cit.*, p. 249.

** It should be pointed out that this method must give too small values for the stresses for the reason that the water is assumed to act on the secondary arch, whereas, it acts on the original arch which has a larger radius. Moreover, the arch does not crack, as indicated by Mr. Mensch; this can readily be verified by computing the eccentricity of the thrust from Professor Cain's formulas. This is $\frac{M}{P}$ and whenever it is less than $\frac{t}{6}$, the section is under compression throughout. Whenever tension, and, consequently, cracking, occurs in an arch, the section is no longer constant, as assumed, and the stress determination is very complicated. Therefore, whenever the secondary arch departs considerably from the original arch, the stresses found in the secondary arch may be much smaller than the actual stresses in the arch. Incidentally, it will be appreciated that it may be dangerous to increase the up-stream radius beyond what computations show that it should be.

cracks developed, they might not permit the water to flow through them and, moreover, the outside skin might not crack due to the swelling of the water-soaked concrete. If the arch under consideration is near the foundation, the shear transmitted to the foundation must also be considered. It is true that the swelling of the up-stream face tends to diminish the tension there, and the time factor tends to equalize the stresses. Both these influences, however, are quantitatively unknown and had better be neglected.

As generally computed by the cylinder formula, the arch stresses are worthless and untrustworthy for the purpose of dam design, although many good arch dams have been designed and dimensioned according to that formula. It is evident that tests are badly needed in order that more economical dams may be designed.

In conclusion, the writer wishes to thank those who have participated in the discussion.

THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI
A SYMPOSIUM
Discussion*

By F. E. MATTHES, ASSOC. M. AM. SOC. C. E.

F. E. MATTHES,† ASSOC. M. AM. SOC. C. E. (by letter).‡—In 1906, while the writer was assisting the late Dr. G. K. Gilbert of the U. S. Geological Survey in research on the transportation of débris by running water, he had occasion to familiarize himself with the details of the German hydraulic laboratories and with the nature of the experiments conducted therein. At that time, those laboratories had been in operation only a few years, yet the results obtained were impressive both because of their practical and their scientific value. Since that time the writer has looked forward to the erection of similar laboratories in this country, specially designed to serve the purposes of "river science". The plans for a National Hydraulic Laboratory as set forth by Mr. Freeman§ are, therefore, of great interest.

No one at all conversant with the results accomplished in Germany can entertain any serious doubts as to their utility. The experiments made during the pre-war period have shown the potentialities of the laboratory method in practical studies, such as those for the design of spur-dikes, groynes, jetties, bridge piers, and for the effective placing of such structures. What is more important, they have demonstrated the possibility of determining the fundamental laws of stream flow and channel building, and of clarifying those theoretical questions always debated by engineers. Prominent among those questions are, for instance, (1) whether a river flowing over sedimentary material remakes its bed completely with each flood; (2) at what stage and by what method the deeps at the bends and the cross-over bars near the points of inflection are formed; (3) in what lines, with respect to the axis of the stream, the sediment moves; (4) why, in a straight channel, deeps form alternately near each bank, but not in the middle; and (5) why, in such a channel, both deeps and bars migrate down stream, whereas, in a curved channel, they remain fixed.

Prior to 1905, these questions had been dilated on—in the elaborate German fashion—by various engineers who had made observations on full-sized rivers. A single series of judiciously planned experiments by Professor Engels at the Dresden laboratory‖ proved more valuable than all these learned discus-

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* Continued from December, 1923, *Proceedings*.

† Geologist, U. S. Geological Survey, Washington, D. C.

‡ Received by the Secretary, November 3, 1923, and published by permission of the Director of the U. S. Geological Survey

§ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1185.

‖ *Zeitschrift für Bauwesen*, Vol. 55 (1905), pp. 664–680.

sions, and, in a few weeks, afforded better insight into these matters than all the observations on large streams.

It is significant that many of these questions are still being debated by American engineers, thereby justifying Mr. Freeman's comment on the slow progress of river science in this country. Within the last few decades, there has been plenty of river engineering in the United States, but little intensive research with a view to developing river science. For one reason most of the rivers with which engineers have to deal offer complex and unsatisfactory subjects for analytical study. Wonder has been expressed that more contributions to river science have not been made by engineers employed on the Mississippi, but the truth is that the "Father of Waters", turbid, turbulent, and a mile in width, is, because of its extreme size, the poorest subject imaginable for systematic observation and economical study. The observer in a laboratory has to deal with a stream only a few feet wide, of clear water, the bed of which he can scrutinize minutely, and every fluctuation of which he can control at will. Manifestly, he has tremendous advantages, and, therefore, much should be expected of him.

The writer does not share the skepticism so often expressed as to whether it is possible to reproduce in a laboratory all the phenomena of flow in a large river such as the Mississippi. An acquaintance with the German experiments and observations on streams and stream beds, both large and small, lead him to expect that some phases of reproduction of phenomena will be much more difficult than others, but that none will be found altogether impossible. The most troublesome, doubtless, will be the processes whereby the Mississippi has built out its delta, including the Passes and the mouths, which processes are associated with the slow settlement and still slower compaction of the colloidal material and fine silt that are carried in suspension. Even these features, however, may be observed occasionally in miniature in brooks and rivulets, and, therefore, it may be inferred that they also can be duplicated under proper laboratory conditions. The physics of colloids is to-day barely out of its cradle and no one would attempt to foretell what research and experimentation in that field may yet accomplish.

As for the actively "meandering" course of the Mississippi between Cairo and Baton Rouge, there can be no serious difficulty in reproducing it in the laboratory, any more than there has been in the case of European rivers. In this part of its course, the Mississippi behaves characteristically as an overloaded, aggrading stream. It picks up its load of sediment when in flood, drops most of it as its volume diminishes, rebuilds its bars, and, deflected by them, develops oxbow loops, or "meanders". All the processes and phenomena which it exhibits, have been observed in brooks and rills, the fundamental laws evidently being the same in large and in small streams.

The difference in the character of the Mississippi above Baton Rouge and below that point, is a matter that has been scarcely touched on in the Symposium, nor so far in the discussion. The engineers best acquainted with the Mississippi may regard it as too well known to need mentioning, but the writer suspects that the main reason for its being passed over is that

it is essentially a physiographic fact such as engineers are not wont to consider. To the writer, who is an engineer turned physiographer, this change in character looms up as a fact of tremendous importance that must be considered in any comprehensive discussion of plans for flood control or rectification. For whatever may be done to change the character or behavior of the river in the upper stretch, is also likely to affect the remainder, yet there are excellent reasons for leaving the lower stretch as nearly as possible in its present general condition, as far as that is satisfactory, and this will be demonstrated briefly.

The outstanding characteristic of the river's course from Cairo to Baton Rouge is its instability or excessive meandering. In past times, it has shifted its bed back and forth, and has occupied innumerable positions within a broad belt—the meander belt, as it may be appropriately termed. Man's principal concern, to-day, is to put a stop to this shifting—a problem more difficult even than that of confining the flood waters between levees.

Below Baton Rouge or, more properly, from the vicinity of Plaquemine downward—for there is really a gradual transition—the outstanding characteristic of the river's course is its inherent fixedness. Here, for a distance of about 250 miles, the river flows practically at sea level, between mere mud walls which were subject to overflow until they were leveed, yet it appears to have maintained its course substantially unchanged for thousands of years. It makes many bends, some of them quite sharp, yet it displays no tendency to enlarge them to oxbow loops. The waters flow with a marvelous economy of energy through this crooked channel of their own making, and appear fully competent to carry their load of sediment to the sea, although the surface slope ranges from only a few hundredths to a few thousandths of a foot per mile, according to the stage. There is no need of dredging, for the river maintains its channel sufficiently deep—100 to 200 ft.—nor is there need of elaborate bank revetment, although the banks consist largely of fine sand and silt. All that was necessary to make the banks permanently inhabitable was the erection of low levees.

Clearly, it is because of the existence of these unusual, almost ideal conditions along the Lower Mississippi that New Orleans has been able to prosper, even though situated at the apex of a sharp bend; and it is equally clear that the future safety and prosperity of the city will depend on the careful maintenance of these conditions.

Unfortunately, the lower course of the Mississippi from Plaquemine to the Head of the Passes, has never been intensively studied from the physiographer's point of view, and, consequently, is not yet fully understood; but this much seems certain to the writer, that it is not susceptible of being changed or improved to any notable degree. It appears to lack the adaptability to new conditions of flow that is possessed by the unstable, meandering course above Baton Rouge, and probably cannot be made to carry appreciably greater flood discharges in the future than at present. This is because the channel cross-section and the relations of depth to width are already such as to provide, with the available slope to the sea, a near approach to a maximum efficiency of discharge at flood stage. Furthermore, the river cannot be induced to

increase materially its channel capacity by scouring, for this could be accomplished only by raising its flood level considerably between correspondingly high levees; but, as Mr. Klorer* has pointed out, a considerable increase in levee height is precluded by the small supporting power of the soil, the limit of which is now almost reached. It is important, moreover, to note that no material increase in the capacity for flood discharge has been brought about by the greater flood heights produced by the progressive completion of the levee system, the effects hoped for having been practically nullified by the lessened slope to the Gulf, due to the extension of the levees down stream. The principle relied on by the Mississippi River Commission, that an alluvial stream, given enough time, will scour out a channel to accommodate its flow, although unquestionably sound in its generality, appears, therefore, not to be applicable to the case of the Lower Mississippi.

It follows that, inasmuch as the completion of the levee system is demanded for the protection of the lowlands up stream, some other expedient must be resorted to along the lower river in order to dispose effectively of the excess flood waters which the main channel cannot carry off. The river itself has pointed the way to the solution. If it is not to be permitted to rise over its leveed banks and find relief by overflow, then it will burst through at some point of weakness and discharge through a crevasse. The logical plan to dispose of the excess flood waters hereafter is, apparently, by means of properly designed and judiciously placed spillways.

To some engineers, the spillway plan has always seemed to be a mere "expediency measure" of doubtful virtue, but it is not at all improbable that, in the end, the flood problems of the Lower Mississippi will be solved by an expediency measure of some kind. It will be essential, however, in any event, that the decision to adopt such a measure shall be reached through a mature consideration of all the facts involved, the physiographic as well as the purely hydraulic. Nothing would be more helpful in this connection and nothing would afford a better insight into the ultimate nature of the effects that are likely to result, than a series of judiciously planned experiments in a hydraulic laboratory.

Thus far, only the question of the disposal of excess flood waters has been considered. In addition, as already pointed out, the stretch between Cairo and Baton Rouge is encumbered with immense quantities of coarse sand. It is pertinent to consider what will happen to this heavy load when the river at last is entirely confined between levees and compelled to carry off its entire flood discharge, without relief by lateral overflow. Will this load of sand, because of the increased transporting capacity of the confined flood waters, be carried farther and farther down stream, thereby upsetting the relatively orderly and tranquil flow of the waters in the lower course?

Mr. Freeman suggests that the travel of the sand grains down the river should be studied for the purpose of ascertaining whether any large proportion of them are ground up into finer products and thus carried in suspension out to the Gulf, or whether they lodge and accumulate in the meandering part

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1159.

of the river's course, thereby gradually raising the bed. Is it to be inferred that these questions have not yet been investigated, sufficiently, at least, so that it is clear whether or not aggradation of the coarser materials has been taking place on any large scale in the section above Baton Rouge? From the physiographer's point of view the question is a vital one, for, if aggradation is and has been taking place normally on a large scale above Baton Rouge, and if the lower limit of this depositional work has not hitherto progressed noticeably down stream, it seems entirely probable that hereafter, as a result of the increased transporting power of the confined flood waters, the coarser materials will be carried farther down stream and the unfortunate shoaling and meandering tendencies to which they give rise, will be imparted progressively to the lower river. The distance from Baton Rouge to New Orleans is so great that no immediate apprehension need be felt about the menace at the latter point; nevertheless, the sooner the problem is studied, the better. Here, again, physiographic investigations in the field, supplemented by laboratory experiments, would be of value.

Several other matters might be considered as pertinent in this connection, but the writer believes that the foregoing will suffice to emphasize the inherent physiographic nature of the main problems presented by the Mississippi, and the consequent need of collaboration between the engineer and the physiographer in devising plans for their solution; and the fact that these investigators should have at their disposal the facilities of a specially designed laboratory.

In conclusion, the writer desires to express the hope that there will be no inclination on the part of engineers to resent his suggestion that the physiographer be consulted and given a share of the responsibility in making the Mississippi safe and useful to the nation. All the phenomena of stream flow and channel building, all the features of a river's bed, banks, flood-plain and delta are physiographic. What else are they? Physiography is that branch of geology which deals with the origin and development of the surface features of the earth. Pre-eminent among the agencies that are active in modeling the land is running water in the form of streams. It follows that of necessity the physiographer devotes a considerable share of his attention to the activities of streams and the consequences thereof. The special problems that arise from the interference of the action of a river with the purposes of civilization are, therefore, properly in his province.

Providing that a National Hydraulic Laboratory is established in accordance with Mr. Freeman's suggestions, it would be desirable to have on its staff not only a physicist competent to deal with the phenomena of molecular physics, but also a physiographer versed in stream flow, channel building, and sediment transport, to whom a river would be not merely a mass of water flowing in an open channel, approximately in accordance with empirical formulas, but something akin to a great, living, pulsating organism, the complex functioning of which is conditioned by many geographic and geologic factors, and which has been evolving for many ages. The Mississippi will never be thoroughly understood until it is studied from this angle. Humphreys

and Abbot long ago realized this, and went to considerable trouble to gather all the geologic information that was available in their day. Unfortunately, geologic science in the middle of the Nineteenth Century was still quite immature and could benefit the investigators but little. Practically nothing was known of the great ice sheets which in recent times, considered from a geological standpoint, covered the northern part of the continent, and it had not been considered in how far the great volumes of water from these ice sheets were instrumental in fashioning the valley of the Mississippi above Cairo and its broad flood-plain below that point. Neither was there an adequate conception of the slow "diastrophic" rising and sinking movements that are now known to affect the crust of the earth, nor had the Mississippi region been studied with especial reference to the effects produced in consequence of movements of this type. As for physiography, or geomorphology, as it is sometimes, and perhaps better, called, it came into existence a quarter of a century later, with the classical studies of Powell, Gilbert, Shaler, Russell, and Davis. However, now that physiography, glaciology, and the other branches of earth science here concerned, are approaching maturity, there is reason to expect that they will be able to contribute much to the understanding of the Mississippi and its valley.

Some investigations, the writer learns, have recently been made in the Mississippi Basin by a well known geologist at the request of the Chief of Engineers of the Army. Whatever their nature or scope may be, it is hoped, that they will mark the beginning of a comprehensive geologic and physiographic survey of the entire Basin.

TENTATIVE SPECIFICATIONS FOR STEEL HIGHWAY BRIDGE SUPERSTRUCTURE

SUBMITTED AS A PROGRESS REPORT OF THE SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

Discussion*

BY MESSRS. L. M. SANFORD, O. H. AMMANN, S. TIMOSHENKO, G. B. WOODRUFF. D. B. STEINMAN, A. W. CARPENTER, and HENRY B. SEAMAN.

L. M. SANFORD,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The writer offers the following observations relative to the Tentative Specification for Steel Highway Bridge Superstructure,§ as presented by the Special Committee on Specifications for Bridge Design and Construction.

Combined Stresses.—In Article 121, the Committee specifies that members subject to combined stresses shall be proportioned so that the combined fiber stress will not exceed the unit stress given in Article 201. In the Specification for Steel Railway Bridge Superstructure,‖ the corresponding clause states that the combined fiber stresses shall not exceed the unit stress "for tension" given in Article 201.

The Railway Bridge Specification would appear to allow 16 000 lb. per sq. in. on a compression member if it is also subject to bending. This seems to be a manifest absurdity, inasmuch as the allowable unit stress on a long column might be as low as 8 000 lb. per sq. in., whereas the addition of a small eccentric or bending load would make 16 000 lb. the allowable unit fiber stress.

In the clause under discussion, the allowable unit stress might be taken as the allowable bending stress or, alternatively, as the allowable compressive stress, and, to this extent, it appears to be ambiguous. On the other hand, if engineers are guided by the earlier specification of the same Committee, they are adopting a clause that appears to invite trouble and certainly would be open to criticism.

Unit Stresses.—The writer believes it would be preferable to use the phrase, "compression members", instead of "compression on columns". The use of the limiting value for a fiber stress of 14 000 lb. per sq. in. on compression members in accordance with the prevailing practice seems to be expedient and a simplication for the designer as compared with the use of a limiting value corresponding to $\frac{l}{r} = 40$.

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* Continued from December, 1923, *Proceedings*.

† Pres., The Rochester Nash Co., Inc., Rochester, N. Y.

‡ Received by the Secretary, November 7, 1923.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1377.

‖ *Loc. cit.*, January, 1923, p. 53.

For special steels having a higher yield point than that of standard structural steel, the specification provides that, in the column formula, the fractional part of the denominator shall be increased in proportion to the higher yield point. The fractional part of the denominator is unquestionably $\frac{l^2}{13\,500\,R^2}$. If this is increased, it would naturally increase the denominator as a whole and this, in turn, would decrease the allowable unit fiber stress. This result is doubtless quite contrary to the intention of the Committee.

Compression Flanges of Beams and Girders.—The formula given in Article 203, $16\,000 - 150\,\frac{l}{b}$, for the unit fiber stress in beams and girders, discloses a rather absurd relationship. If $l = 6$ ft. 1 in., and $b = 6$ in., the allowable fiber stress is 14 175 lb. per sq. in., whereas, if $l = 5$ ft. 11 in. and $b = 6$ in., the allowable fiber stress becomes 16 000 lb. per sq. in. In other words, a change of 2 in. makes 1 825 lb., or more than 11%, difference in the allowable unit stress. It is known that material does not act in any such arbitrary manner, and it would seem to be possible to adopt a better rule than the one given. The Fleming rule of $19\,000 - 250\,\frac{l}{b}$, when $\frac{l}{b}$ exceeds 12, with a maximum of 16 000 lb. per sq. in., would appear to be an improvement.

Rigid Members.—The writer believes that the word, "preferably", in Article 304, might be omitted with advantage.

Net Sections.—The Committee's formula for net sections, given in Article 306, may be theoretically accurate and may be of advantage in the investigation of unusual details. For general practice, however, it has several disadvantages. It is improbable that the detailer would obtain the exact pitch and gauge anticipated by the designer, and the added complication is an objection.

As an alternative, the writer suggests the following:

"Where the longitudinal rivet spacing is less than four rivet diameters, all holes the center to center distance of which longitudinally is less than the transverse distance, shall be deducted to obtain the net section."

This rule, although not as accurate as that of the Committee, is used with ease and is more likely to be observed. It is believed that it would be sufficiently accurate for ordinary construction.

Maximum Length of Rivet.—The writer believes that the older specification that rivets the grip of which exceeds four diameters shall be increased 1% for each additional $\frac{1}{8}$ in. of grip with a maximum grip of six diameters, is not improved on by the specification, given in Article 308, which allows rivet grips of six diameters, but makes no deduction for their length.

Latticing of Compression Members.—The rule as given in the Carnegie "Pocket Companion" to the effect that latticing shall be proportioned to resist shearing stress equal to 2% of the direct stress would be on the safe side, amply accurate for most purposes, and a considerable simplification over the formula given by the Committee in Article 317.

Article 337.—The Committee might improve on its wording where it speaks of an "inclined grade".

Compression Members.—For low trusses it would appear to be advisable to reduce the allowable unit stresses more than that evidently intended by the Committee in Article 204. Unless a vertical truss member is extremely rigid, it does not give full and complete lateral support for the top chord and a deduction similar to that given by the old New York Central Railroad specification for through plate girder spans might be an improvement.

Camber.—The writer believes the Committee's specification in regard to camber is rather unfortunate. If an ordinary highway bridge is to be cambered, it would appear best to produce such camber by a change in the geometric shape of the truss rather than by the method proposed in Article 340. The proposed method is conducive to unfair holes and open joints at the splices as the dead load is small in proportion to the live load, and the dead load deformation is not sufficient to bring the truss to its uncambered shape.

General.—It might be well to amplify the specifications, making provision for proper skid rails to prevent a car or truck hitting a truss member.

On deck spans, the simple provision of a curb 8 in. high is not a sufficient safeguard to prevent reckless drivers from climbing the curb and crashing through the hand-rail.

O. H. AMMANN,* M. AM. SOC. C. E. (by letter).†—The tentative specifications for steel highway bridges proposed by the Special Committee on Specifications for Bridge Design and Construction, again bear evidence of the thorough manner in which American engineering practice is being unified and standardized. In so far as they aim at uniformity of strength and serviceableness, these specifications cannot be endorsed too strongly, and it is to be hoped that, through co-operation with other National associations, which have already compiled specifications covering the same field, further unification may eventually be obtained.

It appears opportune, however, to call attention to the danger of too far-reaching standardization when it affects features of design in which the judgment, ingenuity, or taste of the engineer should and may have full sway, as long as he adheres to those features which assure the required strength and rigidity. "Standardization has become a dangerous adversary of progress", is the recent warning sounded by one of the foremost modern scholars.

Such features as type of bridge to be used for certain spans may well be omitted from the specifications or modified so as to permit of broader interpretation, the more so as there are types available other than those mentioned, even for comparatively small highway bridges.

Likewise, the minute prescription of type and arrangement of lateral and sway-bracing does not appear to be justified, in view of the fact that other systems or arrangements have proved to be effective.

The depth ratio of girders and trusses is another feature which does not assure what is intended, namely, a uniform minimum degree of rigidity. Even

* Cons. Engr., New York, N. Y.

† Received by the Secretary, November 7, 1923.

for a given depth ratio, the rigidity, as measured by the deflection, depends greatly on the quality of material, ratio of live load to dead load, and type of truss, so that it would be more appropriate to prescribe limiting values for the deflections and allow the designer to assume the proportions accordingly.

Although not specifically stated, the proposed specifications are evidently limited to highway bridges of ordinary size and type and for such they will obtain good proportions. In accordance with recognized principles of design, they would be wasteful for spans longer than about 300 ft., or even for shorter spans accommodating more than two lanes of traffic.

As suggested by the writer in his discussion* of the Committee's Specifications for Steel Railway Bridges, there is no reason why specifications cannot be written, which will embrace bridges of all sizes. It may be properly argued that larger bridges are few and should be left to the specialist. There is no reason, however, why a committee of experts of the Society should not state the fundamental requirements which will cover the design of bridges in the broadest sense and thus also obtain much desired uniformity of strength for larger bridges. Such requirements imply the adoption of high unit stresses, such as those generally recognized as proper for large bridges, and the specified live load and impact are to be selected so that, for small bridges, they will cover, with a good margin for future increase, the heaviest actual loads and their impact effects, but that with increasing width and span length, they will decrease commensurate with the decreasing density and average intensity of the load combinations under probable conditions. This decrease in the specified load should be much greater than that provided for in the proposed specifications, more particularly in combination with the low unit stresses, considering the fact that, on larger bridges, the highway traffic will ordinarily not exceed an average of 10 lb., and rarely, if ever, 25 lb. per sq. ft.

While engaged in recent years in the design of some large bridges with Gustav Lindenthal, M. Am. Soc. C. E., the writer has had occasion to make a thorough study of individual loads, load combinations, and actual loads on large bridges, together with their effect on various parts of such structures. As a result, a simple load specification was written which, with uniformly high unit stresses, resulted in a consistent proportioning of all parts affected by the widest range of load, from a single concentration to a combination of 34 lanes of traffic of various kinds and more than 3 000 ft. in length. It would lead too far to quote these specifications, and it is not the writer's intention to recommend at present radical changes in the proposed specifications. They have to be used for the field they cover, but it appeared to be advisable to point out their limitations and the feasibility of specifications of a broader scope.

The following suggestions for a modification of the loading specifications are made, therefore, in connection with the unit stresses as proposed.

The proposed live load is unnecessarily cumbersome for conception and application. The writer would prefer a multiple of the same load on each lane of traffic, and similar or proportionate loads for different classes of bridges.

* *Proceedings*, Am. Soc. C. E., April, 1922, p. 912.

The loading proposed by Mr. Harold D. Hussey* appears to be simple and quite suitable, and its designation by a class index, as has become common with railway loadings, is particularly commendable. The only objections which the writer has to that loading are that it resembles too much a uniform train load with an engine at the head and that the calculation of stresses in all spans is affected and complicated by the concentrations, unless elaborate tables of equivalent uniform loads are available. Furthermore, the loading, as proposed, does not make allowance for the decreasing intensity of load with increasing width of bridge or number of lanes of traffic.

In order to overcome these objections, the writer would recommend the following modified loading specification:

For Class A bridges, on each lane of traffic, 10 ft. wide, a loading, which may be designated as T-25, consisting of either a single 25-ton truck or a uniform load of 250 lb. per sq. ft. multiplied by the following coefficient of reduction for length of load, l:

$$K = 0.2 + \frac{80}{100 + l}$$

This uniform load amounts to 200 lb. for $l = 33$ ft., 150 lb. for $l = 100$ ft., and 100 lb. for $l = 300$ ft., and approaches asymptotically 50 lb. per sq. ft.

For more than one lane of traffic both this single truck load and the uniform load should be multiplied by the following coefficient of reduction for number of lanes of traffic, n:

$$C = 0.5 + \frac{2}{3 + n}$$

This reduction amounts to 10% for 2 lanes, 23% for 4 lanes, and 32% for 8 lanes, and approaches asymptotically 50 per cent. With this loading, only spans up to about 60 ft. are affected by the concentrations; for longer spans only uniform loads have to be considered. The stresses may be calculated for the same load and finally multiplied by the coefficients previously given.

For Class B and C bridges, the writer would recommend proportional loadings, say, T-20 and T-15, respectively. If the width of roadway is less than a multiple of 10 ft., the load may be reduced proportionally but, if possible, not less than 10 ft. should be allowed per lane, in order not to impair the speed or the safety of the traffic.

The specified impact allowance is probably as good as any which may be proposed, in view of the incomplete data available, but, again, the writer would recommend a reduction, if the number of lanes of traffic, n, is greater than 1, by multiplying by a coefficient, $C_i = \frac{4}{3 + n}$. This reduces the specified impact to 80% for $n = 2$, 57% for $n = 4$, and 36% for $n = 8$, and approaches asymptotically zero.

S. Timoshenko,† Esq. (by letter).‡—In this discussion the writer will consider only those parts of the proposed specifications in which the question of

* "Proposed Loading for Highway Bridges", *Proceedings*, Am. Soc. C. E., August, 1923, p. 1301.

† Research Dept., Westinghouse Elec. & Mfg. Co., East Pittsburgh, Pa.

‡ Received by the Secretary, November 8, 1923.

elastic stability is involved. The proper solution of this question for steel structures, especially for those of steel of greater strength than that of structural grade, is of great importance because with increase in the strength of the material, the cross-section of the members and the thickness of the plates can be diminished and a combination of conditions conceivably may be established from which failure will occur through sidewise buckling of the compressed members or plates rather than through the flow of material under direct stress. In order to emphasize the importance of these points, the following suggestions are made.

The simplest and most important problem involving elastic stability is the design of columns. In order to determine the working stress, in pounds per square inch, of members composed of structural steel, the well known Rankine formula:

$$\frac{16\,000}{1+\frac{l^2}{13\,500\,r^2}} \qquad (1)$$

has been proposed. In addition,

"For members composed of steel of greater strength than structural grade, the allowable stresses may be increased in proportion to the higher yield point of the stronger steel, provided the yield point is not more than 70% of the ultimate strength. In the column formula, the fractional portion of the denominator should be increased in the same ratio."

In considering this adjustment, it is necessary to distinguish between two classes of compression members, the wide ones and the slender ones, the first failing by flow of metal, directly compressed, and the second by flexural stresses due to the sidewise deflection of the compressed member.

It may be expected that, for members of the first class, the proposed increase of the allowable stress will be in proportion to the increased ultimate strength of the member; but it is rather difficult to justify the proposed increase for the case of slender columns. It is well known in the latter case that sidewise buckling may occur at stresses which are below the elastic limit and that the ultimate strength of a column depends only on the magnitude of the modulus of elasticity.

Consider, for instance, two columns of the same slenderness ratio, $\frac{l}{r}=120$, one of structural steel and the other of steel of greater strength with, say, twice as high a yield point. Assuming that the ends can rotate freely when deflection occurs, it is easy to show that, in both cases, sidewise buckling will occur at compressive stresses below the elastic limit; and inasmuch as the moduli of elasticity of the two materials are practically equal, the ultimate stresses will also be equal.

The proposed column formula gives for the second column an allowable stress 32% greater than for the first; on the basis of the analysis given, this does not appear to be justified.

In the case of built-up columns, so extensively used in steel structures, the resisting capacities are always less than those of solid columns with the same value of $\frac{l}{r}$ and depend on details such as lattice-bars and spacing plates, or battens.

Consider, for instance, the case of Fig. 17.* The ultimate strength of this latticed column is less than that for a corresponding solid column in the ratio,†

$$\frac{1}{1 + P_e \dfrac{1}{\sin \alpha \cos^2 \alpha \, E \, A_d}} \qquad \ldots\ldots(2)$$

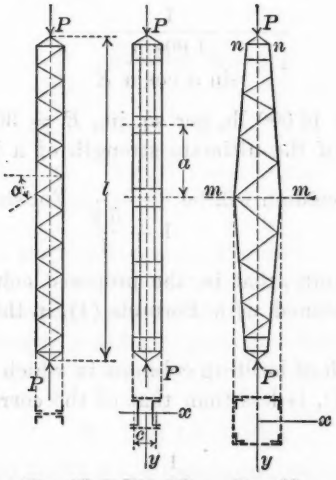

FIG. 17. FIG. 18. FIG. 19.

in which,

E = the modulus of elasticity;

A_d = the area of cross-section of lattice-bars;

$$P_e = \frac{E \, I \, \pi^2}{l^2} \qquad \ldots\ldots(3)$$

which is the ultimate load according to the well known Euler formula for solid columns with freely rotating ends, I being the moment of inertia about the neutral axis.

It is easy to note that, in those cases where insufficient latticing has been provided, the ultimate strength of the latticed column will be far below the strength of the solid column that has the same value of $\frac{l}{r}$‡.

If the latticing is proportioned in such manner as to resist a shearing force equal to,§

$$R = \frac{P_e \, l}{4\,000 \, y} \qquad \ldots\ldots(4)$$

* The formulas given herein, and the numerical tables, except Table 8, are from the writer's papers on elastic stability published in the *Bulletin* of the Polytechnical Institute, in Kiew, 1907–10. For a French translation of one of these papers, see, *Annales des Ponts et Chaussées*, 1913. The writer's papers on the same subject, published in German, appear in *Zeitschrift für Mathematik und Physik*, 1910, and in *Der Eisenbau*, 1921.

† It is assumed that the ultimate stress is below the elastic limit.

‡ The insufficient latticing of compressed members was considered a primary cause of failure of the first Quebec Bridge.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1388, Article 317, of the Tentative Specifications.

then the area of cross-section of the lattice will be:

$$A_a = \frac{P_e\, l}{4\,000\, y\, p\, \cos\alpha} \qquad (5)$$

in which p denotes the working stress.

By substituting Formula (5), in Formula (2), the following ratio is obtained:

$$\frac{1}{1 + \dfrac{4\,000\, p}{\sin\alpha\, \cos\alpha\, E}\ \dfrac{y}{l}}$$

Take, for example, $p = 16\,000$ lb. per sq. in., $E = 30\,000\,000$ lb. per sq. in., and $\alpha = 30°$, the ratio of the ultimate strength of a latticed column to that of a corresponding solid column will be $\dfrac{1}{1 + \dfrac{5\, y}{l}}$. For slender columns, this ratio differs very little from unity, that is, the proposed column formula of Article 201* (Formula (1)), combined with Formula (4), in this case gives satisfactory results.

The ultimate strength of built-up columns in which are used spacing plates or battens only (Fig. 18), is less than that of the corresponding solid column in the ratio:

$$\frac{1}{1 + P_e \left(\dfrac{a\, c}{12\, E\, I_2} + \dfrac{a^2}{24\, E\, I_1} \right)} \qquad (6)$$

in which,

a = the distance between the axes of battens;
c = the distance between centers of gravity of channels;
I_1 = the moment of inertia of cross-section of a channel about the axis through its center of gravity and parallel to the y-axis; and
I_2 = the moment of inertia of the cross-section of a batten about the horizontal axis through its center of gravity.

Numerical Example.—For instance, in the case of a cross-section consisting of two 15-lb., 10-in., channels,† assume that $l = 30$ ft., $a = 36$ in., $C = 7.3$ in., the thickness of battens $= \frac{3}{8}$ in., and the width of battens, 9 in. Then, the moment of inertia of the cross-section, calculated as for a solid column, is $I = 123$, $I_1 = 2.3$, $I_2 = 45.6$ (all in inches to the fourth power).

Substituting in Formula (6), it will be found that the ultimate strength of the built-up column is equal to 0.82 of the ultimate strength of a corresponding solid column.

Compression members of variable cross-section will now be considered. (Fig. 19.)

Let I_m denote the moment of inertia of the cross-section, m-m, and I_n the moment of inertia of the cross-section, n-n. The ultimate strength of such a

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1384.
† Pocket Companion, Carnegie Steel Co., 21st Edition, p. 297.

member will be less than that of a column of constant cross-section with a moment of inertia, I_m, and can be calculated as for a member of constant cross-section with smaller moment of inertia, I_c, determined by the formula: $I_c = f_0 I_m$. In this case, f_0 is a numerical factor, less than unity, the magnitude of which depends on the value of the ratio, $\frac{I_n}{I_m}$. Some numerical values of this factor are given in Table 8.

TABLE 8.

$\frac{I_n}{I_m}$ =	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
f_0 =	0.545	0.645	0.713	0.770	0.815	0.860	0.897	0.932	0.965

By using Table 8 in conjunction with Formula (1), the working stresses for a compressed member of variable cross-section can be easily calculated.

In the case of half-through truss spans, in Article 326* of the Tentative Specifications, it is proposed, in order to prevent sidewise buckling of the compressed top chord, to proportion the vertical truss members and the floor-beam connections by the formula:

$$R = 150\ (A + P) \qquad (7)$$

It is rather difficult to verify this formula analytically, because the ultimate strength of a compressed top chord depends on its flexural rigidity and on the resistance to deflection of the vertical truss members, rather than on the area, A, of the cross-section of the chord or on the strength of the vertical members as assumed in Formula (7).

Consider, for instance, the simplest case of a through truss span of a constant height, h, having a uniformly distributed load, q (Fig. 20). The compressive force at the middle of the top chord will be equal to $\frac{q\,L^2}{8\,h}$. The ultimate strength of the top chord can be obtained from the usual column formula. It is only necessary to substitute for the length, L, a reduced length, l, such that,

$$l = f_1 L \qquad (8)$$

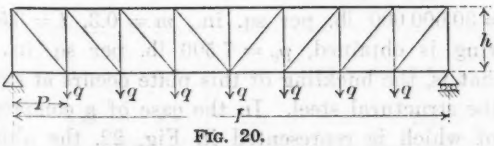

FIG. 20.

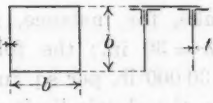

FIG. 21.

FIG. 22.

The magnitude of the factor, f_1, depends on the flexural rigidity, EI_v, of the top chord in a horizontal plane (I_v being the moment of inertia of the top chord about a vertical gravity axis), and on the resistance which the vertical members offer when buckling of the top chord occurs. This resistance is a

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1389.

function of the quantity, $k = \frac{Q}{P}$, in which Q denotes the lateral force that, if applied to the vertical member only at the top chord panel point (neglecting for the moment, the horizontal resistance of the chord), produces a lateral deflection of the vertical member equal to unity. Assume that the rigidity, EI_v, of the top chord is constant; likewise Q, and, therefore, k. Then, using the factor, $\frac{K L^4}{16 E I_v}$, as argument, the coefficient, f_1, of reduced length can be calculated analytically. Some numerical values of the coefficient are given in Table 9.

TABLE 9.

$\frac{K L^4}{16 E I} =$	0	5	10	15	22.8	56.5	100	163	200	300	500	1 000
$f_1 =$	0.694	0.524	0.443	0.396	0.363	0.324	0.290	0.259	0.246	0.225	0.204	0.174

By using Table 9, the reduced length, l, can be calculated from Formula (8) and afterward the working stress can be found from Formula (1), as for a simple compressed member of length, l.

In the design of compression members of large cross-sections, the question of buckling of compressed plates may be of practical importance. Consider, for instance, a compressed member the cross-section of which is given in Fig. 21. By diminishing the thickness of the column wall, the member will fail under compression by the sidewise buckling of plates themselves and not by the deflection of the column as a whole. The sides may be considered as long rectangular plates with free supported edges. The ultimate strength of such plates in compression depends on the ratio between the thickness of the plate, t, and its width, b. Let

$$p_e = \frac{\pi^2 E t^2}{12 (1 - m^2) b^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which, m denotes Poisson's ratio. Then, the ultimate stress, p_u, in the case considered will be:

$$p_u = 4 p_e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Take, for instance, $E = 30\,000\,000$ lb. per sq. in., $m = 0.3$, $t = 0.5$ in., and $b = 30$ in.; the following is obtained, $p_e = 7\,500$ lb. per sq. in., and $p_u = 30\,000$ lb. per sq. in., that is, the buckling of this plate occurs at stresses within the elastic limit of the structural steel. In the case of a compression member, the cross-section of which is represented in Fig. 22, the ultimate strength against buckling can be calculated by using the formula for a long rectangular plate with one longitudinal edge free and the other built in. This stress is represented by the formula:

$$p_u = \frac{4}{3} p_e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Assuming, for instance, that $t = \frac{1}{2}$ in. and $b = 20$ in.; the following is obtained, $p_e = 16\,900$ lb. per sq. in., and $p_u = 22\,500$ lb. per sq. in. With a decrease in the ratio, $\frac{t}{b}$, the magnitude of p_u decreases in the proportion, $\frac{t^2}{b^2}$, and becomes, in the case considered, almost negligible. In order to increase the ultimate strength, it is necessary to rivet stiffener angles along the free edges of the webs.

With respect to the strength against buckling of the web of a plate girder, the required thickness of the web is dependent on the maximum shearing stress. Such webs are usually of insufficient thickness to prevent sidewise buckling and, therefore, require the substitution of a heavier web, or the addition of stiffeners. Usually, these changes are made without considering the stability of the stressed web, and types of construction with widely varying factors of safety result. For example, the plate girders of the New York Connecting Railroad, with a span, $L = 100$ ft., and a height, $h = 10$ ft., have a web thickness, $t = \frac{5}{8}$ in., whereas, in the case of the Canadian Pacific Railway, with nearly the same height, the web thickness equals $\frac{7}{16}$ in. near the supports and only $\frac{3}{8}$ in. at the middle. These two plate girders, both satisfying American specifications, have very different factors of safety with regard to the buckling of the web.

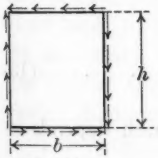

FIG. 23.

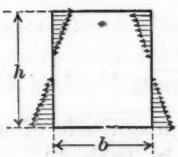

FIG. 24.

More satisfactory results can be obtained from a consideration of the stability of the stressed web. Near the supports, the shearing force is the most important factor; therefore, the part of the web between two stiffeners can be considered as a rectangular plate with supported edges, subjected only to the action of shearing stresses (Fig. 23).

The critical value of the shearing stress, s, can be calculated from the formula:

$$s_u = f_2\, p_e \qquad (12)$$

in which, p_e has the same meaning as before* (Formula 9), and f_2 is a numerical factor depending on the ratio, $h : b$. Some values of this coefficient are given in Table 10.

TABLE 10.

$\frac{h}{b}$ =	1	1.2	1.4	1.5	1.6	1.8	2.0	2.5	3.0
f_2 =	9.42	8.0	7.3	7.1	7.0	6.8	6.6	6.3	6.1

Assuming that $E = 30\,000\,000$ lb. per sq. in., $m = 0.3$, and $b = 5$ ft., the critical values of shearing stresses for different values of the height, h, are as given in Table 11.

* It is assumed that $h > b$.

Using a factor of safety, 2, it is easy to show, by Table 11, that, in both examples of the plate girders, previously mentioned, a thickness of ½ in. will insure the stability of the web near the supports.

TABLE 11.—CRITICAL VALUES OF SHEARING STRESSES IN PLATE GIRDER WEBS HAVING STIFFENERS SPACED 5 FT. APART, IN POUNDS PER SQUARE INCH.

h, in feet.	t, IN INCHES.			
	⅜	7/16	½	9/16
5	9 980	13 600	17 700	22 400
7	7 730	10 500	13 700	17 400
10	6 990	9 510	10 400	15 700

It is assumed that the buckling of the web is not accompanied by bending of the stiffeners, which assumption will be true only when the stiffeners have a sufficient flexural rigidity.

The necessary magnitude of the moment of inertia, I_3, of the stiffeners about a gravity axis parallel to the web can be calculated from the following approximate formula:

$$I_3 = f_3 \frac{b\,t^3}{12\,(1 - m^2)} \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (13)$$

in which, f_3 denotes a numerical factor, the magnitude of which depends on the ratio, $b:h$. Some values of f_3 are given in Table 12.

TABLE 12.

$\frac{b}{h}$ =	1.0	0.75	0.625	0.50
f_3 =	3.3	11.6	25.2	60

Assuming, for instance, that $b = 5$ ft., $h = 10$ ft., and $t = \frac{1}{2}$ in., the following is obtained:

$$I_3 = 60 \times \frac{60}{8 \times 12 \times 0.91} = 41 \text{ in.}^4$$

At the middle of the span, considering normal stresses only, the part of the web between two stiffeners will be in the condition represented in Fig. 24.

Assuming that the rectangular plate has freely supported edges, the critical value of the maximum normal stress will be:

$$p_u = f_4\ p_e \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (14)$$

Some values of the coefficient, f_4, depending on the magnitude of the ratio, $b:h$, are given in Table 13.

TABLE 13.

$\frac{b}{h}$ =	0.40	0.50	0.60	0.667	0.75	0.80	1.0	1.5
f_4 =	4.65	6.40	8.68	10.6	13.5	15.6	25.6	54.3

Computations will show that the critical stress varies little with the ratio $b:h$, and that the placing of stiffeners close together at the middle of the span will not affect materially the stability of the web.

In practice, the critical stresses will be somewhat greater than those obtained from the given formulas, due to the rigid fastening of the edges (it was assumed that the edges were simply supported). Nevertheless, it is probable that, in some cases, buckling of the web occurs under unfavorable loading conditions. This buckling does not represent an immediate danger to the girder, but it does cause an over-stressing of the girder flanges and of the flange rivets near the stiffeners—an undesirable condition.

G. B. WOODRUFF,* M. AM. SOC. C. E. (by letter).†—Referring to the loads and unit stresses, as proposed by the Special Committee on Specifications for Bridge Design and Construction, the writer believes it would be desirable to increase the unit stresses by 40% and the live loads in the same proportion. It is uneconomical and, therefore, faulty engineering, to design for the heavy dead loads of present-day highway bridges at the unit stress of 16 000 lb. per sq. in. Such requirements lead to making the floor construction, on which the life of the structure largely depends, as light as possible, whereas good design demands the opposite.

The detail loadings proposed in the specification are cumbersome to apply and should be simplified.

In connection with a 40% increase in unit stresses, the writer would propose a 30-ton truck followed by a uniform load of 1 000 lb. per lin. ft. for each lane of traffic. In order to care for the permissible reduction in unit loading with increasing widths of roadway, the formula, $K = \frac{1}{2} + \frac{W}{40}$, is suggested, in which K is the proportion of stresses from one lane of traffic carried by one truss, and W is the width of roadway, in feet.

Article 7.—Depth Ratios.—It is difficult to understand the difference between the requirements for plate girders and rolled beams. A ratio of $\frac{1}{15}$ for plate girders should be permitted.

Article 8.—Clearance.—In order to allow for the overhang of wide truck bodies, the clearance line should be back not less than 12 in., and preferably 15 in., from the curb.

Article 101.—Dead Load.—A large proportion of highway bridges now being built have only a concrete roadway surface. On such bridges, a wearing surface of some type will probably be required in the future. In order to

* Brooklyn, N. Y.

† Received by the Secretary, November 17, 1923.

care for this contingency, it might be desirable to specify an excess dead load of 25 lb. per sq. ft. of roadway for bridges with such floors.

Distribution of Truck Wheel Loads.—This section is very complete, but it would be desirable to make it more concise by the use of formulas. It should be extended to cover distribution of wheel loads through concrete slabs.

Article 113.—Impact.—The term, l, should be replaced by nl, in which n is the number of lanes of traffic on the bridge. It is hardly logical to provide for more impact in a floor-beam hanger than is provided in the floor-beam itself. The idea of using extra material in the hangers is excellent, but such provision should be made in another manner, preferably by considering the hanger and the floor-beam as a continuous frame.

Article 123.—Secondary Stresses.—The last sentence of this Article is ambiguous. It should be omitted or prescribe in detail what secondary stresses should be considered.

Unit Stresses.—The writer prefers the straight-line compression formula and believes that, within the prescribed limits of $\frac{l}{r}$, the knowledge of column behavior does not warrant any closer refinement than is obtained by that formula.

Splices.—The requirement regarding full strength splices for compression members in Article 312, is rather severe. As far as the writer is aware, no trouble has been experienced with properly designed "50%" splices.

Extra Rivets in Packing.—Article 313 is very indefinite. The wording of the similar clause in the A. R. E. A. Highway Specifications is preferable.

Article 322.—Lateral Bracing.—Bottom laterals should not be required in deck plate girder spans less than 50 ft. in length. The allowable spacing of cross-frames should be increased to 18 ft. in order to correspond with the allowable 6 ft. for stiffener spacing.

Girder Cover Plates.—The necessity of bottom cover-plates extending the full length of the girder, is not apparent to the writer. The specification for additional length of cover-plates is faulty, as it leads to such close rivet spacing as to cut the section more than necessary. There is no reason for the covers to be fully developed at the theoretic end. The clause calling for an excess length of not less than 18 in. is to be preferred.

End Bearings.—The length of span for sliding bearings should be extended to 90 ft. Rollers are an expensive detail, and unless they receive more than the usual maintenance, they generally become so dirty and rusty that they fail to work and a sliding bearing results. Where rollers are necessary, 6 in. should be the minimum diameter.

In conclusion, the writer would urge that an effort be made by all parties interested in bridge specifications to "get together." Three highway bridge specifications, differing only in minor details, appearing at such close intervals, cannot be considered as a hopeful sign for unity of the profession.

D. B. Steinman*, M. Am. Soc. C. E.—The need for standardization of highway bridge specifications has long been felt. The Special Committee on

* Cons. Engr., New York, N. Y.

Specifications for Bridge Design and Construction has done a highly creditable work in producing the Tentative Specification, but a number of its features might be improved.

Perhaps the section most open to criticism is that covering "Live Loads." This division of the Tentative Specification leaves much to be desired in respect to clearness and simplicity, particularly from the viewpoint of those who have to interpret and apply the clauses in the actual work of design.

The Committee could not do much better than to adopt the general plan of highway loading specification recently presented to the Society by Mr. Harold D. Hussey.* A loading system of this type would possess distinct advantages over the Committee's loading specifications; it would offer greater simplicity, ease of application, and generality of scope. Therefore, the speaker would recommend the adoption of Mr. Hussey's general system of loading, with such modifications as may be made.

Following Mr. Hussey's system, with some modifications, the speaker would recommend a specification providing for four different loadings, namely, T-25, T-20, T-15, and T-10; the index number in each case designating the weight, in tons per truck of the loading represented. This would furnish a range sufficient to cover the requirements of practically all the different highway bridges, from the heaviest city bridges to the light structures ordered for export. The unit roadway width, or width per line of trucks, as proposed by Mr. Hussey, might well be revised from 9 ft. to 10 ft. Accordingly, the T-25 loading would consist of axle concentrations of 10 kips and 40 kips, respectively, followed by a uniform load of 1 000 lb. per lin. ft. per 10-ft. roadway width; T-20 would consist of axle concentrations of 8 kips and 32 kips, respectively, followed by a uniform load of 800 lb. per lin. ft.; T-15 would consist of axle concentrations of 6 kips and 24 kips, respectively, followed by a uniform load of 600 lb. per lin. ft.; and T-10 would consist of axle concentrations of 4 kips and 16 kips, respectively, followed by a uniform load of 400 lb. per lin. ft. The load values and, consequently, the stresses, for the different classes of T-loading would be directly proportional to the index numbers of the respective classes; so that a single standard table or graph of equivalent uniform loads could be used for all classes of T-loading.

Another modification that may be suggested for the Hussey loading diagram is the addition of a uniform loading in front of the truck concentrations, this advance uniform load being of the same intensity as that which follows the axle concentrations. This modification would have the following advantages:

1.—It would afford a better representation of actual conditions on bridge roadways, where any local concentration of heavy trucks is generally preceded as well as followed by vehicles of lighter weight.

2.—It would facilitate the calculation of stresses, inasmuch as the load position for maximum bending moments will always be given by placing the heavier axle concentration at the center of moments.

* "Proposed Loading for Highway Bridges", *Proceedings*, Am. Soc. C. E., August, 1923, p. 1031.

3.—It would reduce the disparity between the equivalent uniform loads for maximum moments and shears in the same spans.

For spans exceeding 300 ft., the speaker would disregard the axle concentrations of the T-loading system, and advocate the use of the respective uniform loading as a continuous load. Accordingly, for the longer spans, the T-25 loading would be 1 000 lb. per lin. ft. per traffic lane; T-20 would be 800 lb. per lin. ft.; T-15 would be 600 lb. per lin. ft.; and T-10 would be 400 lb. per lin. ft.

In order to provide for the reduced probability of full loading on spans of increasing length, the speaker would advocate a clause specifying an appropriate reduction of assumed loading for longer spans. For this purpose, he would suggest a reduction of the specified loading by 1% for each full 100 ft. in length of span of more than 300 ft. This rule would give a reduction of 4% for a 705-ft. span; 12% for a 1 500-ft. span; and 25% for a 2 800-ft span.

In the manner outlined, complete, unambiguous, consistent provision can be made for four (or more) different classes of loading intensity and for all spans.

In order to provide for different widths of roadway, Mr. Hussey's plan of multiplying the stresses calculated for a unit lane of traffic by a coefficient representing the number of traffic lanes in the roadway width can be followed. Accordingly, if the unit width (one traffic lane) is taken as 10 ft., the multiplier, or width coefficient, for any width, W, would be:

$$K = \frac{W}{10} \qquad (15)$$

In the speaker's opinion, modification appears to be desirable in this feature in order to provide for a reduction of intensity of loading with increase of width, taking into consideration the reduced probability of full loading on a larger number of lines of traffic. When the width of roadway reaches or exceeds a value of about 40 ft., there is much less likelihood of all the lines of traffic being fully loaded at the same time. To take this fact into account, the speaker would suggest retaining Equation (15) for all widths up to 40 ft., and, for greater widths, using a formula corresponding to a line of flatter slope, say,

$$K = 2 + \frac{W}{20} \qquad (16)$$

for all widths exceeding 40 ft. This rule would give $K = 4$ for a 40-ft. roadway; $K = 5$ for a 60-ft. roadway; $K = 6$ for an 80-ft. roadway; $K = 8$ for an 120-ft. roadway; and $K = 10$ for an 160-ft. roadway.

Instead of the foregoing rule, the suggestion made by Glenn B. Woodruff, M. Am. Soc. C. E., to use a single formula for all widths* might be adopted, namely:

$$K = 1 + \frac{W}{20} \qquad (17)$$

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1913.

This rule would give $K = 3$ for a 40-ft. roadway; $K = 4$ for a 60-ft. roadway; etc., up to $K = 9$ for an 160-ft. roadway. The only difficulty with this rule lies in its application to roadway widths of less than 20 ft. For instance, at a width of 10 ft. (corresponding to a single line of traffic), the coefficient, K, would become $1\frac{1}{2}$, but even this might be desirable in order to penalize the adoption of narrow traffic widths. The speaker, however, would prefer the specification represented by Equations (15) and (16), as that would make it unnecessary to take into account any reduction formula for the usual widths of highway bridges (less than 40 ft.).

For spans having three or more trusses or girders, and for those carrying cantilevered roadways, the width, W, in the preceding equations is understood to mean the width of roadway required to be loaded to produce maximum stress in a respective truss or girder.

The Tentative Specification appears to be limited in scope in the matter of roadway widths. No suitable provision has been made for widths greater than 18 ft., and no adequate consideration has been given, apparently, to the much greater widths provided in many recent designs. The foregoing suggestions for width coefficients indicate how variations in roadway width may be consistently covered in the specifications, without abandoning the fundamental conception of lanes of traffic for the less progressive procedure of specifying a constant uniform load per square foot.

The Tentative Specification is also ambiguous in the provision of a fourth class of bridges, Class D, to carry electric railways, together with an unspecified highway live load. It would be clearer and more consistent to segregate the classifications of highway and electric railway loadings, and to describe a combination bridge as designed to carry a specified truck loading together with another specified trolley loading. Accordingly, the graduated classes of highway loading might be designated as Classes A, B, C, and D, respectively; and a specified series of electric railway loadings might be designated as Classes L, M, and N, respectively; so that the loading for a combination bridge would be definitely described by a designation such as A-L or D-N. Instead of using these alphabetical class symbols, it might be preferable, for greater clearness and flexibility, to use numerical indices of loading classes, such as T-25 to T-10 for truck loading and L-50 to L-25 for electric railway loading; in each case, the numerical index expresses the weight of the unit vehicle, in tons. On this basis, a loading specification of—T-20, L-40—would designate a combination bridge designed for 20-ton trucks and 40-ton trolley cars. Fig. 3* in the Tentative Specification would be a loading diagram for L-40.

The impact formula proposed by the Committee makes no provision for reduction of impact from multiple lines of traffic. Such provision should and could easily be made by adopting an impact formula having in its denominator the factor, n, in which n denotes the number of lines of traffic. The specification should also differentiate between the impacts on different kinds of roadway, such as paved floors, plank floors, and railway tracks, by using

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1381.

a formula having in its numerator a coefficient varying with the character of the roadway.

The sections on materials in the Tentative Specification include the standard specifications of the American Society for Testing Materials for structural carbon steel and for structural nickel steel. The speaker wishes to take this opportunity of advocating the inclusion in the bridge specifications of the Society of a section covering silicon steel which has supplanted nickel steel as the most efficient alloy for long-span bridges. Silicon steel gives nearly the same strength as nickel steel, at a much lower cost. Manufacturers and designers of long-span bridges have practically eliminated nickel steel from consideration as an economical alloy, and have substituted other high strength materials, the chief of which is silicon steel. In a number of large structures built recently, and in a number of designs for structures soon to be constructed, silicon steel has been adopted as the principal material. Thus far, no standard set of specifications has been written for silicon steel. There is urgent need for an authoritative specification covering this material, and an excellent opportunity is presented for the Committee to adopt a specification covering silicon steel, to be added to the sections covering the other structural materials in the Tentative Specification.

A. W. CARPENTER,* M. AM. SOC. C. E.—It is common knowledge that two other Societies are working on specifications for the design of highway bridges. It would be very unfortunate if three different sets of specifications were formulated. The other Societies are the American Railway Engineering Association and the American Association of State Highway Officials. It is suggested that the Committee, unless it has already done so, co-operate with these other bodies and agree on all essential points as far as possible.

With reference to specific points in the Tentative Specifications presented by the Committee, the speaker has one comment as to the suggestion of a minimum width of 12 ft. for a highway bridge. It seems a mistake even to suggest such a dimension. Although the 12-ft. minimum is intended for one line of traffic, it seems certain that at times vehicles would attempt to pass on that width and that disaster would result. It is suggested that a width of approximately 16 ft. be considered as the absolute minimum for highway bridges.

The unit stresses proposed are based on the usual standard of 16 000 lb. per sq. in. in tension, which figure is being criticized extensively as too low; this is a valid objection, in the speaker's opinion, unless it is thought that the loadings will increase in the future beyond those specified. The future development of loads is difficult to foretell, but should be, and no doubt has been, the subject of much study by the Committee.

The speaker notes the lack of provision for designing in material other than steel (except cast iron in a minor capacity). No working unit stresses are given for timber, which, of course, is used extensively for joists and floors. Also, there are no clauses covering pavements of any kind, nor any regarding the design of concrete floors, or other concrete parts. In regard to timber floors,

* Asst. Valuation Engr., N. Y. C. & H. R. R. R. Co., New York, N. Y.

it is desirable to present a specification for construction that will not rattle under traffic.

HENRY B. SEAMAN,* M. AM. SOC. C. E.—There may be one or two clauses of the Tentative Specification for Steel Highway Bridge Superstructure, which were an oversight, such as Article 202 which limits the length of columns to 100 or 120 times the least radius of gyration. The Rankine column formula is applicable to columns of indefinite length, and should not be restricted except to require a rigid member which would not vibrate under passing loads. This limit might be placed at $\frac{l}{r} = 200$, or even greater. In an important bridge, recently examined, the length of columns was equal to $\frac{l}{r} = 185$, and these columns remained perfectly rigid under passing loads.

For the maximum unit stress to which a short column of structural steel may be subjected, the Committee first adopted 14.0 kips; but when it was realized that any steel of greater strength required a different limit, it was decided to make the clause general in its application. It was placed, therefore, at the value for $\frac{l}{r} = 40$, which is more correct, and applies to all steels. For stronger steels, the fractional part of the denominator of the Rankine formula is also changed, with the numerator, because, for very long columns, the strength must merge into the Euler formula.

As the allowable stress in columns is limited to the value for $\frac{l}{r} = 40$, so also should the allowable stress in compression flanges of girders be limited to the value for $\frac{l}{b} = 12$.

The restriction of the length of rivets to six diameters, in Article 307, is based on the experiments made some years ago by the Erie Railroad Company. Four series of tests were made on: First, rivets the holes for which were rough-punched; second, rivets with holes punched and reamed; third, rivets, hand-driven; and, fourth, rivets, machine-driven. It was found in all cases that up to six diameters the rivets filled the hole, but not beyond that length. If a rivet fills the hole, it will resist full stress; if it does not fill the hole, it should not be subjected to any stress. The clause sometimes used, permitting a graduated stress for long rivets, therefore, seems to be untenable.

* Cons. Engr., New York, N. Y.

RAILROAD TRANSPORTATION AND RAILROAD TERMINALS

A SYMPOSIUM

Discussion*

By J. G. Roney, Assoc. M. Am. Soc. C. E.

J. G. Roney,† Assoc. M. Am. Soc. C. E. (by letter).‡—It is unfair to compare the railroads of the United States with the brewing and distilling industry because the two enterprises have nothing in common and their aims and purposes are diametrically opposed. The former have for their purpose the serving and benefiting of the public, which is supposed to bring a fair return to the security holders, whereas, practically no aim can be ascribed to the brewing and distilling interests than that of making money through the degradation and debauchery of mankind, except, of course, the manufacture of alcohol for industrial purposes. The railroads are not in the same category as breweries and distilleries, and it seems to be out of place to suggest the abolition of the former. The suppression of the beer and liquor-making industries was brought about as a natural consequence of right prevailing over wrong, because their products were harmful as well as unnecessary, whereas the railroads are necessary for the nation's existence and welfare.

The greatest trouble with the railroads to-day is not lack of regulation, but too much regulation. Conditions like those under which the railroads are compelled to operate would be fatal to almost any other industry. In order to function properly, the railroads should be permitted to manage their own affairs to a greater extent, without so much interference from outside agencies. Under the present system, outside agencies determine how much the railroads shall pay their employees, decide what they shall receive in freight rates, set a limit on their earnings, and place many other restrictions. This outside interference is not confined to the Federal Government alone, but comes also from the governing bodies of each State, county, city, town, and village through which a railroad may pass. The greatest degree of railroad regulation occurred during the recent period of Federal Control and, as this maximum regulation proved to be such a failure, it is only just that a period of minimum regulation should follow, in order to demonstrate the economy and efficiency with which a railroad may be operated when permitted to manage its own affairs.

It has been unjustly charged that the Government took over the operation of the transportation systems because the railroad executives could not operate them. The real reason was that the extraordinary times demanded like measures, one of which was the consolidation of all the railroads of the country

* Continued from November, 1923, *Proceedings*.

† Asst. on Engr. Corps, Pennsylvania System, Pittsburgh, Pa.

‡ Received by the Secretary, November 7, 1923.

into one vast system, for the single purpose of transporting men and materials needed in the World War. Under the existing laws, it was not possible for the railroads to effect this consolidation, and it became necessary for the Government to do that which was not legal for the railroads to do. A single fact which shatters the charge of inability, is that the executives were retained by the Government to operate the railroads through the period of stress.

From time to time, the Detroit, Toledo and Ironton Railroad has been set up as a model of railroad operation. It is generally conceded that this railroad is, more or less, an adjunct to the Ford Motor Company, compared with which, it is of much less importance. If each of the railroad systems of the nation might have as a feeder a vast, prosperous industry intimately connected with it, railroad rates might be abolished and free transportation instituted. Granting that the Detroit, Toledo and Ironton Railroad has been efficiently operated and placed on a paying basis, its strategic location and the peculiar conditions under which it operates, however, make it an unfair criterion for the transportation systems of the United States. It would be ideal if all the vast mileage of the railroads might be operated under conditions similar to those surrounding the Detroit, Toledo and Ironton, the length of which is only about 450 miles, an infinitesimal part of the total American railroad mileage.

PERIODIC FLUCTUATIONS OF RAINFALL IN HAWAII

Discussion*

By Frederic I. Winslow, M. Am. Soc. C. E.

Frederic I. Winslow,† M. Am. Soc. C. E. (by letter).‡—This paper and its deductions present what is probably the strongest case yet shown regarding the correlation of solar phenomena and precipitation.

A similar presentation which appeared about twelve years ago in a French scientific journal, and abstracts of which were published in American magazines, did not make a case sufficiently strong to suggest the possibility of forecasting rainfall, which, after all, would be the real test of an ascertained correlation.

In order to assemble certain scattered data bearing on the subject, the following quotations§ are given:

"The question of cyclical variation of rainfall coincident with sun-spots has been very fully discussed by Blanford, so far as India is concerned. He points out the peculiar fascination such theories exert over many minds, and he emphasizes the necessity for rigorous scrutiny of all such cycles by a general array of facts. He quotes from Russell in New South Wales to show that cycles for periods of two, three, five, six, nine, ten, eleven, twelve, thirteen, seventeen, nineteen, thirty, and fifty-six years have been brought forward, with a large amount of *prima facie* evidence.

"Blanford examined the rainfall of all India for twenty-two years, and in arranging the rainfall in the biennial, triennial, etc., up to quinquennial series, he 'found that the cyclical series could always be brought out, but this amplitude proved nothing very different from the probable error of the average.' After elaborately discussing the rainfall of India as a whole for two complete sun-spot cycles, Mr. Blanford says: 'It may therefore be confidently concluded that the total rainfall of India, exclusive of that of Ceylon and the Burmese peninsula and (of course) of the seas around, affords no evidence whatever of an eleven year periodical variation.'

"An examination of the annual mean anomaly of the total rainfall of India for twenty-one years (1864 to 1885), indicates that such variations are accidental in India at least.

"A comparison of the rainfall of separate provinces shows that in 1871, a maximum sun-spot year, the precipitation of the Konkan was 14.6 inches deficient, while that of Malabar was 2.4 inches in excess. In ten years only out of eighteen did the four rainfall provinces of India have the same sign to its annual deviation.

"A similar theory as to the prevalence of droughts in the years of minimum sun-spots, and of heavy rain at the sun-spot maximum has found many supporters. Blanford has compared the record of sun-spots and droughts, and

* Discussion on the paper by Joel B. Cox, Assoc. M. Am. Soc. C. E., continued from December, 1923, *Proceedings*.

† Cons. and Div. Engr., Metropolitan Water-Works, Framingham, Mass.

‡ Received by the Secretary, November 10, 1923.

§ "American Weather," by A. W. Greely, 1888.

makes the definite statement that there is no 'dependence of the one class of phenomena on the other,' since the record shows that not only do droughts occur in India at other times than at the sun-spot maximum, but even 'sometimes in years of maximum sun-spots'."

Gen. Greely further quotes from Blanford as to the theory that the temperature of the air is subject to variations running in periods of about eleven years in inverse order to the number of sun-spots, stating that Blanford examined the temperature of India for thirty-one years (1850–1880), and says, "It is evident that there is no indication whatever of an eleven-year period or any other in the 'temperature anomalies' of India." Gen. Greely himself examined the following representative stations with reference to the influence of sun-spots on precipitation: San Francisco, Calif., St. Louis, Mo., Marietta, Ohio, Troy, N. Y., and Gardiner, Me., from 1857 to 1887, Cheyenne, Wyo., from 1870 to 1886, and Omaha, Nebr., from 1870 to 1887, these stations being

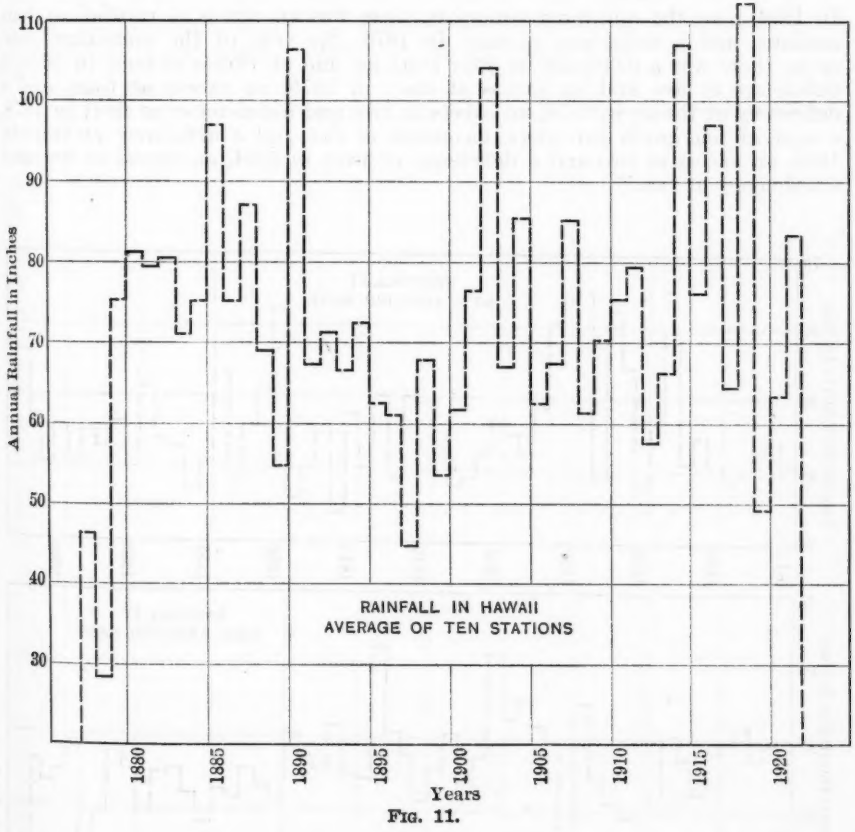

FIG. 11.

chosen on account of their widely varying longitudes and their climates representative of the whole of the United States. Gen. Greely states further that:

"There was only one year in which the departures of these stations had the same sign (1864), when there was a deficiency of rainfall at five stations.

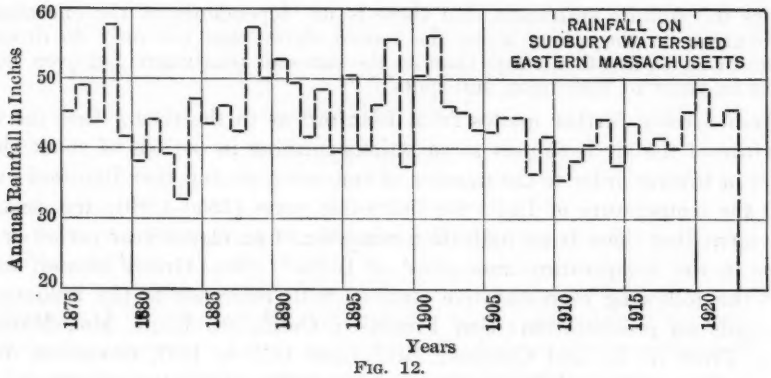

FIG. 12.

In 1866, near the minimum sun-spots, there was an excess of rainfall at four stations, and a deficiency at one; in 1870, the year of the maximum sun-spots, there was a deficiency at four stations, and an excess at one; in 1874, a deficiency at five and an excess at one; in 1875, an excess at four, and a deficiency at three; in 1876, an excess at two and a deficiency at five; in 1878, a year of maximum sun-spots, an excess at five and a deficiency at two; in 1880, an excess at two and a deficiency at five; in 1884, an excess at five and a deficiency at two."

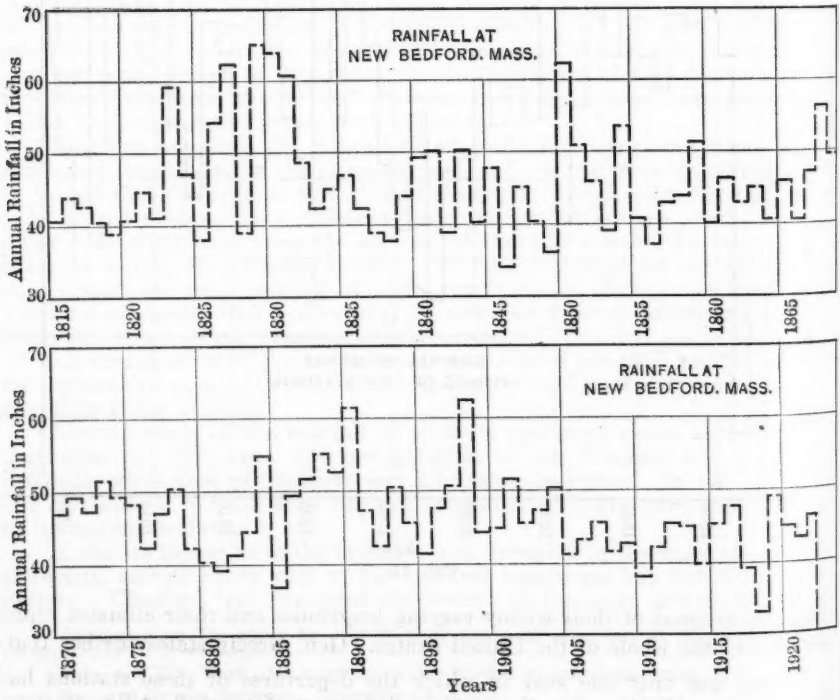

FIG. 13.

These facts clearly demonstrate, in the opinion of Gen. Greely, that any apparent connection between the amount of rainfall and the periodicity of sun-spots was entirely accidental. A diagram of the rainfall variation in Hawaii, plotted from the author's data, is shown in Fig. 11, also one covering the Sudbury water-shed in Massachusetts, from 1875 to 1922, in Fig. 12, and one, Fig. 13, showing the records at New Bedford, Mass., supposedly covering the longest consecutive period in the United States (from 1814 to 1922). In the last two cases, it would be extremely difficult to say the least to posit any law covering the amount of rainfall. It would seem likely, therefore, that only in cases similar to that produced by Mr. Cox, namely, where the area is surrounded by water, would it be possible to trace any connection between rainfall and sun-spots.

Adherents of the idea that sun-spots bear certain relations to the frequency of thunder-storms* and that the weather may be foretold by the observation of solar phenomena,† are not uncommon. Another writer, W. T. Foster, of Washington, D. C., also concludes that solar disturbances are accompanied by terrestrial atmospheric disturbances.

* *Cosmos*, Paris, 1914.

† "Long-Range Weather Casting and Its Methods," by J. S. Ricard, *Popular Astronomy*, April, 1911.

PROGRESS REPORT OF THE SPECIAL COMMITTEE TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.

Discussion*

By Messrs. Jacob Feld and Herbert Chatley

Jacob Feld,† Jun. Am. Soc. C. E. (by letter).‡—As the writer is much interested in the subject of soils, he wishes to express his appreciation of the valuable data included in the Progress Reports of this Committee for 1922§ and 1923.‖ In correlating the study of soils with the field of colloid chemistry, the Committee again shows the necessity of a broader application of the natural sciences to civil engineering design.

The importance of the colloid constituents in soil has been appreciated for several years by men working in applied colloid chemistry. Unfortunately, the Civil Engineering Profession has been largely out of touch with the modern advances in colloid chemistry and has not applied the results to the design of structures in which the action of soils is an important factor. It is hoped that the recommendations of the Committee will cause a change in the attitude of the civil engineer.

In the discussion of the writer's paper on the "Lateral Earth Pressure"¶, Henry Goldmark, M. Am. Soc. C. E., contributed some field tests of the soil at the new ship lock in New Orleans Harbor. The material showed typical colloidal properties. A practical knowledge of the facts in the Committee's reports should aid greatly in the control of this soil.

The purification and clarification of water is another application of colloid chemistry. The writer has pointed out** how the electrical properties of the colloidal constituents explained the mutual precipitation of the impurities in two different waters mixing in a settling basin.

At the same time, it must be remembered that colloid chemistry is far from an absolute science, and that much research is necessary to establish the laws definitely. It is hoped that the opening of a new group of applications will stimulate the work required.

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* This discussion (of the Progress Report of the Special Committee on Bearing Value of Soils for Foundations, etc., presented at the Annual Meeting, January 17, 1923, and published in *Proceedings* for October, 1923), is printed in *Proceedings* in order that the views expressed may be brought before all members.

† Brooklyn, N. Y.

‡ Received by the Secretary, November 8, 1923.

§ *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 523.

‖ *Loc. cit.*, October, 1923, Papers and Discussions, p. 1732.

¶ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1553.

** *Engineering News-Record*, November 30, 1922, p. 946.

In its Progress Report for 1921,* the Committee included an exhaustive treatment of the colloidal properties of clay, but did not treat of another important colloidal constituent of the soil, namely, the humus. In the following notes, the writer will enumerate some of the more important general properties and applications of the various colloids in soil.

Soil is a porous mass of hard framework plastered over with a jelly containing chemically active matter, plant foods, unstable organic compounds, etc., the pores containing air and water. When a substance is said to be in the colloidal state with respect to another substance, it means that it is dispersed or subdivided through the other substance (usually water) to form particles of molecular to microscopic dimensions. When in this state, substances possess certain characteristic properties, and as soil exhibits these properties it may be classed as a colloid. These phenomena may be enumerated as: Adsorption; size of particles; coagulation or sedimentation; Brownian movements; and action with water, acids, bases, and salts.

In 1878, van Bemmelen published his first results on the colloid constituents of soil and their effect on fertility. He stated that:

"The fertility of the soil is proportional to the colloid content. The retentive power of rich soils for the salts necessary to the growth of plants is due directly to the presence of colloid solutions and properties in the humus and the clay of the soil."

In 1868, Warrington had noticed the absorptive properties of soil, chiefly humus.

Soil Constituents.—In order to understand the origin of colloids in soil the methods of soil formation must be studied. Soil is the product of the disintegration and decomposition of rocks, chiefly through the agency of water. The rock particles were broken off and washed into the sea, to be mixed there with particles from other sources as well as with decomposed animal and vegetable life. In time, this accumulation of mixed particles was exposed to the air and then began to undergo a second process of erosion (weathering) and disintegration. In the second step, organic decomposition products aided the natural elements: Air, water, and changes in temperature.

The complex material which is called soil, consists of:

1.—The mineral matter derived from the original rock, consisting of potassium, calcium, and aluminium silicates, constituting the framework of the soil. This part is largely unalterable, although it contains some re-active products of decomposition.

2.—The calcium carbonate and phosphate and organic matter derived from the marine and other organisms deposited in the soil.

3.—The soil water, a dilute solution of carbonic acid, containing some small quantities of soluble soil constituents.

4.—Residues of plants which have grown since the soil occupied its present position, consisting of the mineral matter taken up from the water as food and of part of the complex organic matter.

* *Proceedings*, Am. Soc. C. E., February, 1921, Papers and Discussions, p. 9.

The U. S. Bureau of Soils classifies the various soil constituents as separated by a mechanical analysis into sizes corresponding to the standard sieves, as shown in Table 1.

TABLE 1.

Constituent.	Diameter, in millimeters.	Diameter, in inches.
Fine gravel	2.00–1.00	0.0788–0.0394
Coarse sand	1.00–0.50	0.0394–0.0197
Medium sand	0.50–0.25	0.0197–0.0098
Fine sand	0.25–0.10	0.0098–0.0039
Very fine sand	0.10–0.05	0.0039–0.0020
Silt	0.05–0.005	0.0020–0.0002
Clay	Below 0.005	Below 0.0002

NOTE.—The standard No. 200 sieve has an opening of 0.074 mm., or 0.0029 in.

The coarser aggregates are chemically inert. The clay and the finer silt constituents, however, act differently from the remainder; there is also a difference in chemical composition, chemical, and physical properties. Top soil is 2.06% organic matter, by weight, and the subsoil is 0.83%, as determined by Schreiner and Reid.

Soil Colloids.—The colloids in soil may be classed as follows: Silicic acid, silicates, aluminium and iron hydroxides, humus acid, and clay in general. Experiments made by the U. S. Bureau of Soils, in connection with the irrigation project at Fallon, Nev., show the presence, and the effect, of colloid silica in soil as follows:

"One gram of good soil, shaken in 10 cu. cm. of dialysed colloid silicic acid (sp. gr. 1.0108), coagulates in 3 to 8 hours at 28° cent. One gram of bad soil shaken in the same solution, keeps it as a colloid for a period of more than 10 days, even after the pure silicic acid colloid has coagulated."

Silicic acid causes impermeability. Treatment of bad soils with a salt or an acid causes a loss of this property by the coagulation of the colloid. The use of gypsum in fields is advisable for this reason.

The fertility of the Nile is explained by Pohl in accordance with the van Bemmelen theory. The silt of the Nile contains a large quantity of colloid hydrated silica, which readily absorbs the potassium, nitrogen, and phosphorous compounds from the water. During the spring floods, the food-carrying particles are spread over the farm lands and transport the chemical foods to the soil.

Clay.—Clay is a mixture of hydrated silicates of aluminium, iron, alkali, and alkali-earth metals, mixed with quartz, mica, and feldspar. All grains of clay are coated with a colloidal jelly which contains the hydroxides of aluminium, iron, and manganese, as well as silicic acid and silicates. Too much colloid produces a clay that is called sticky, fat, or strong; too little (for moulding or working), produces one that is lean, sandy, non-plastic, or weak. Clay is formed by the decomposition of feldspar rock through the agency of water. It is formed in the hydro-sol state, which, later, may coagulate. The properties of clay depend largely on its colloid covering, and both iron and aluminium hydroxides, the most important of the colloids, are amphoteric,

that is, they can re-act either as an acid or a base. This will explain the contradictory statements made by different experimenters concerning the colloidal action of clay.

Clay is not a chemical compound. In his investigation of Dutch soils, van Bemmelen found two classes of soil-clay constituents, the ratio of the number of silica to alumina molecules being 3 : 5 and 2 : 1. The former is soluble in dilute hydrochloric acid and the latter only in hot concentrated sulphuric acid. Finding no cases in which the ratio was that of constant whole numbers, he finally called clay an absorption compound of the type: $Si\,O_2$. m $(Al_2\,O_3)$. n $(Fe_2\,O_3)$ .... p $(H_2\,O)$.

Clay is a plastic colloid; on drying, it shrinks and absorbs heat, and, on re-moistening, it swells and gives out heat. When placed in water the separate clay particles show Brownian movements and settle very slowly in spite of the high specific gravity. Clay is an electro-negative colloid, its reaction being due to a trace of potash liberated by hydrolosis. It is flocculated only by a solution containing positive ions, the extent of flocculation increasing with the valence and concentration of the anion used.

The plasticity of clay is caused by the re-solution of the dried out colloids in the clay. In coagulating, clay will carry down, absorb, and retain such substances as oils, fats, soap, starch, maltose, albumen, casein, inorganic substances, like Prussian blue, coal-tar dyes, carmine, blood, all bad odors, etc., hence it is used for decolorizing, clearing, and purifying. Coagulation of the clay in river waters occurs at the deltas near the ocean when the salt solutions in the sea water causes precipitation or coagulation. This explains the richness of soils near the mouths of rivers.

The real importance of clay in soils is its high absorptive powers, which keep all food at the surface instead of allowing it to be washed into the earth. The colloids which it forms around the rootlets act as the homes of the soil organisms.

Humus.—Humus, which is the product of the decomposition of vegetation, consists of the products derived from sugars and proteins, mixed with hydroxides, oxides, and silicates of iron and aluminium. Peat soil is largely composed of humus. The water content (by weight) that various soils will retain is as follows: humus, 10%, humic garden soil, 38%, dry soil, 11%, and fine sand containing no organic matter, 0.2 per cent. Borntraeger's analysis of peat soils showed its composition to be water, 25%; fiber, 50%; humic acid, 22%; and ash, 3 per cent.

Humus has six properties which affect the soil, as follows:

1.—It gives a dark brown or black color to the soil.
2.—It draws various ions, ammonium, phosphate, potassium, etc., from their solutions.
3.—It causes the soil to puff up, increasing the air space.
4.—It increases the water-holding power of the soil.
5.—It swells when wetted, breaking up the soil after rains.
6.—It disappears very slowly from the soil.

In 1888, van Bemmelen published a remarkable paper on the colloid properties of the humus constituent of soil. His researches were followed by Baumann who recognized the following properties:

1.—Very high capacity for holding water.
2.—Extraordinary shrinkage on drying.
3.—Reversibility: The freshly precipitated material re-dissolves when the precipitant is washed away.
4.—Coagulated by acids, salts, electric current, and frost.
5.—Decomposes salts: Calcium carbonate, phosphate, etc.
6.—Forms difficultly soluble and easily decomposible colloid mixtures with other colloids.
7.—Masks certain ion reactions: Iron cannot be detected by the ferrocyanide test in the presence of humus.
8.—Forms adsorption compounds.

Colloid Properties of Soil.—The most important colloid properties of soil as a unit are:

1.—The power of adsorbing substances from solution, for example, fertilizers.
2.—The power of absorbing large quantities of water and holding it for long periods. In time of drought, soil will cause the ascent of water from great depths.
3.—The power of changing from the flocculated to the deflocculated state and *vice versa.*
4.—The power of entering into solution in pure water and of being thrown out of solution by the addition of small quantities of electrolytes, that is, the reversible reaction of sol to gel.

The property of adsorption is important in that it keeps the soluble salts from being washed out. In his report on the "Power of Soils to Absorb Manure", Mr. J. T. Way made a special study to determine which part of the soil was the chief adsorbing agent. He finally concluded that it was the double salts of lime and alumina, sodium and alumina, etc., chiefly the silicates, which were acting as a jelly-like covering over the clay particles. He held the idea that the process of adsorption was purely chemical, the soluble salts forming part of the double silicate compounds.

Wiegner examined all the known experimental data and showed that they all obey the same law: $\frac{y}{m} = K\, C\, \frac{1}{p}$, in which, y is the amount adsorbed by a quantity, m, of the adsorbent; C is the concentration after equilibrium is reached and adsorption is complete; and K and p are constants depending on the solution and adsorbent. The bases adsorbed liberate an equivalent amount of base from the soil. The assumption is that only the basic part of the reagent is adsorbed; the remaining free acid radical combines chemically with the basic part of the soil.

Some experimenters claimed that only the humus gives colloidal properties to the soil. In order to disprove this the following statement is given.

S. Oden* made non-colloidal humic acid and alkali humates from peat. These substances did not dialyze, showed no ultra-microscopic structure, were not precipitated or coagulated by salts, and adsorbed only with difficulty. However, Professor Suzuki, working at the same time and in the same laboratory, prepared humic acids from peat, which were pure colloids, as they showed ultra-microscopic form, were easily precipitated or coagulated by sodium chloride, and easily adsorbed by bone-black.

The property of absorbing water is chiefly characteristic of the humus in the soil. The U. S. Bureau of Soils states that: "The amount of water absorbed by clay and given up at 60° and 130° cent. is proportional to the colloid content." Texas white fire clay gives up 3.08% water at 60° and 3.78% at 130° cent. Keen showed that evaporation of water from soil was not regular and did not follow the laws of evaporation from liquids. However, when the soil was ignited and the colloid structure destroyed, the evaporation became regular.

Chemical Action.—The action of chemicals on soil is either to cause flocculation (coagulation), or to prevent it. The deflocculators are the hydroxides, carbonates, and other highly hydrolized salts of the alkali metals. Concerning the interaction of minerals with water solutions Mr. E. C. Sullivan states†:

"Action of silicates, etc., of the earth's crust on the sols of salts which do not dissolve with alkaline reaction, consists of an equivalent exchange of bases. Certain colloids do take both the acid and basic radicals, but such is not adsorption. These colloids are usually the solids which may exercise both acid and basic functions."

van Bemmelen suggested that the action was purely chemical. He showed that the action of alkali solutions on silicates consists of: First, a reaction between the free alkali and the silicate, forming a substitution product; then the metal displaced from the silicate forms a slightly soluble salt which is precipitated. By extracting the oxides of iron and aluminium from the soil, he removed the colloid silicates and hydroxides, after which there was no precipitation of the carbonate, phosphate, or borate.

In connection with experiments on the action of salts and acids on soil, the U. S. Bureau of Soils gives the following result:‡

"The action of salts on clay depends upon the relative solubility of possible products. A reaction which results in a net decrease in solubility of the components, produces an increase in the degree of flocculation; and a reaction resulting in greater solubility causes deflocculation."

The action of alkalis on soil colloids is to cause deflocculation. The alkali forms an alkali silicate which hydrolizes to form a sol. The addition of more alkali causes an increase in the solution of sodium salt and sodium sol, to a certain point, after which the quantity of sodium in solution causes a drop in the quantity of the sol and coagulation soon results. Ammonium salts cannot cause coagulation, as the ammonium hydroxide is a weak base and cannot set back the hydrolosis. Because potassium salts are more soluble than the

* *Koll. Zeitschrift*, v. 14, 1914, p. 23.

† *Bulletin No. 312*, U. S. Geological Survey, 1907.

‡ *Bulletin*, 1908, New Hampshire Agricultural Experiment Station.

corresponding salts of sodium, their effect should be weaker, and experiment bears this out.

Valence does not affect the coagulation of soil colloids, which perhaps is due to the protective action of the soil colloids, especially the humus. The U. S. Bureau of Soils states that "relative solubility or insolubility of the salts of higher valence is the cause of the faster coagulation by salts of higher valence."

Effect of Salts on Clay.—

Sodium chloride	(Valence 1)....	Precipitates soon after the calcium.
Calcium chloride	(" 2)....	Prompt precipitation.
Aluminium chloride	(" 3)....	No effect.
Tin chloride	(" 4)....	Hydrolized over night and carried down the clay sol.

Effect of Electrolytes.—Excess of potassium hydroxide or chloride added to a potassium-clay soil reduces its solubility and causes a salting out or coagulation, changing the sol to a gel. Addition of hydrochloric acid or calcium sulphate results in a hydrogen or calcium compound which is soluble and forms a gel. This is a case of the exchange of bases. Sodium hydroxide or oxalate added to the hydrogen or calcium gels causes the formation of a sodium gel. This is a case of deflocculation.

Soil is usually acid, but it becomes basic on boiling. This is often assumed to be a test of the presence of carbonic acid. If common salt is added, however, the acidity is increased, which shows that there is a reaction between sodium chloride and clay with the formation of hydrochloric acid.

Fertility of the soil is in proportion to the colloid content. By a careful study and proper regulation of the colloid content, the fertility can be controlled. The methods and results of such control are as follows: Freezing the soil decreases its richness by causing the irreversible coagulation of the colloids; burning the soil increases its richness by causing a precipitation of the foods held by the colloids; and adding lime, which is basic, coagulates the colloids, because both the clay and the humus are negative or acid. In many soils, there is too much clay, and trouble arises during rainy weather. Lime coagulates the clay and prevents the greater part of its absorption of water. Calcium sulphate acts like lime in coarsening the soil; the colloids are coagulated, and the excessive swelling is reduced. Adding calcium sulphate with a phosphate fertilizer is beneficial, because the colloids are prevented from adsorbing the phosphate and the plants receive a greater proportion of the latter. Manure added to soil without lime is largely wasted. Lime should be added first until it is well slaked, when the colloids are coagulated. Then, the manure is placed on the soil, and the plants obtain the full benefit of the fertilizer. Otherwise, the clay will cause the formation of complex salts which are too insoluble to function in a manner beneficial to soil fertility.

In conclusion, soil is made up of several aggregates, the most important of which are colloids. By a proper study of these, the fertility and behavior of the soil can be controlled.

HERBERT CHATLEY,* ESQ. (by letter).†—The colloidal content of clay is undoubtedly of great importance, but there is a limit to the influence it exerts. Pure "China clay" and similar almost wholly colloidal clays are less cohesive and plastic than those in which the colloidal fraction is only, say, 50 per cent. This naturally suggests that, as in concrete, the cohesion and plasticity are related to the grading of the grains and to the geometrical fact that denser packing is possible with unequal grains than with equal ones. Therefore, in a tension-colloid curve, there will probably be a maximum beyond which the tension will decrease with increased colloid. The attention of the Committee should be directed to the Oden clay-analyzer which has been made in an improved form by the Cambridge (England) Instrument Company. This instrument enables the grading to be rather precisely indicated.

Another point which should be considered is that in the colloidal state the "bound" water may rise to more than 50% by volume so that with very fine clays a small amount of stiffness may occur in a slip containing only 7% by volume of clay.‡ Consequently, the properties of soils cannot be simple functions of the moisture content.

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* Engr.-in-Chf.'s Office, Whangpoo Conservancy Board, Shanghai, China.
† Received by the Secretary, December 13, 1923.
‡ Bingham, "Fluidity and Plasticity."

THE RAILWAYS' PART IN THE DEVELOPMENT OF HAMPTON ROADS

Discussion*

By W. D. Faucette, M. Am. Soc. C. E.

W. D. Faucette,† M. Am. Soc. C. E.—As a natural physical water basin, Hampton Roads is probably unexcelled in the Western Hemisphere. It is admirably located, and as explained by Mr. Churchill,‡ it is reached by several large railway systems. The lines controlled and operated by these several railway systems comprise approximately 33 000 main-line miles, or nearly one-eighth of the entire railway main-line mileage in the United States. This gives Hampton Roads direct through-rail connections with that part of the country from the Great Lakes to the Gulf of Mexico and east of the Mississippi River. Those who have read Mr. Churchill's paper will note the rapid progress and the gradual growth of the rail lines and the port facilities at Hampton Roads. As mentioned by the author, the branch of the Seaboard Air Line Railway Company was put in operation in December, 1836, and is now a part of its main system.

The railroad systems, as now constituted, must function properly, or the port facilities which are available at these great Roads are of no avail. The performance and the efficiency of Hampton Roads and its port, as well as any other port facilities, are dependent on the efficiency of those carrier lines that work between the tributary area and these ports. The history of transportation is the history of progress. Transportation is irrevocably bound up and woven into every activity, and the machinery of operation of the carriers is the developing factor of Hampton Roads and of every other American port. The executive heads of these carriers are fully alive to the fact that the inability at any time of these lines to function properly spells slow handling of traffic and slow development of port facilities.

It must be admitted that the railroads are now doing and will continue to do their parts well, and that this country, with all its industrial successes, its port operations, its import and export business, and its great coal-docking facilities, must develop and produce the required results only in direct accord with the service rendered to these port facilities from the distant interior points to the coast. It should be emphasized in no uncertain way that the growth and expansion of the railroads and their allied operations is the index of National progress. Without adequate and economical transportation, failure results. With proper transportation comes success, and to the railroads of

* This discussion (of the paper by Charles S. Churchill, M. Am. Soc. C. E., presented at the Fall Meeting of the Society at Richmond, Va., October 18, 1923, and published in *Proceedings* for December, 1923), is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Seaboard Air Line Ry. System, Norfolk, Va.

‡ *Proceedings*, Am. Soc. C. E., December, 1923, p. 2006.

the United States is due a great debt for the position which this country has taken, and which it will and must maintain.

The investment in railroad property of the United States is estimated roughly at $19 000 000 000. The 33 000 miles of railroad tributary to Hampton Roads Port would cost from $2 000 000 000 to $2 500 000 000 to reproduce. This indicates the magnitude of the plant that contributes to the success and importance of this Port.

This subject is not one in which justice can be done by mere oral discussion. The Society and individual members are indebted to Mr. Churchill for the fund of data and information which he has selected and presented so interestingly.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

THOMAS WHITE COTHRAN, M. Am. Soc. C. E.*

DIED FEBRUARY 9, 1923.

Thomas White Cothran, the son of Wade E. and Sara E. (Chiles) Cothran, was born in Abbeville County, South Carolina, on September 18, 1874. At the age of eighteen, he entered Clemson College from which he was graduated with the degree of B. S. in 1896. Immediately after his graduation, on account of his high scholastic standing, Mr. Cothran was appointed Assistant Instructor in Drawing in that institution. Thus, his first position was attained solely through his own efforts, merit, and fitness, as were all other positions which he afterward held. He never sought influence, preferring to rely on his own resources and ability to win.

Following his work at Clemson College, Mr. Cothran was connected with the U. S. Geological Survey as Draftsman and Levelman in Texas and Iowa during 1897 and 1898, and, subsequently, he made a number of important water-power surveys in North Carolina.

In 1900, Mr. Cothran entered the service of the Seaboard Air Line Railway Company as Chief Draftsman, serving in this capacity for five years. While in this position, he designed many of the Company's important buildings and other structures. As Assistant to the Chief Engineer of the Carolina, Clinchfield, and Ohio Railway, Mr. Cothran obtained experience in the location and construction of that remarkable line through heavy mountainous territory, that was invaluable.

In 1905 and 1906, as Resident Engineer of the Atlanta, Birmingham, and Atlantic Railway, he had charge of much important work under trying circumstances and in difficult locations.

As Principal Assistant Engineer of the Norfolk and Southern Railway, Mr. Cothran had oversight of the construction of this line's Southern Extension to Raleigh, N. C., and other work. Among the structures involved was the great bridge across Albermarle Sound, between Edenton and Mackeys Ferry, N. C., which is approximately 27 000 ft. in length (about 5 miles). This successful design and construction was due largely to Mr. Cothran's foresight and judgment in the preliminary investigations and surveys, his work in this connection having been thorough and complete.

On the conclusion of his service with the Norfolk and Southern Railway Company, Mr. Cothran entered private practice at Greenwood, S. C., in 1908, and, as Consulting Engineer, designed and built a number of important works, among which was the system of sewers and sewage disposal at Rock Hill, S. C.

From 1917 to 1919, he served as a member of the South Carolina Highway Commission, subsequently becoming its Director, in which position his ex-

* Memoir prepared by Philip Aylett, Assoc. M. Am. Soc. C. E.

perience and executive ability proved to be invaluable. The influence of his work is still felt in that Department.

Mr. Cothran is survived by his widow, who was Miss Maud Boswell, of Portsmouth, Va., and six children. He was a capable Engineer and Architect, always energetic and resourceful, and those whose privilege it was to have known him, cherish the memory of his association and true loyal friendship. By his premature death, the Engineering Profession has sustained a real loss, and his friends have suffered a great bereavement.

He was a member of the Baptist Church, a 32d Degree Mason, a Knights Templar, and a Shriner. He was also a member of the Rotary Club. Politically a Democrat, he always reserved his right to independent discrimination in local issues.

Mr. Cothran was elected an Associate Member of the American Society of Civil Engineers on July 9, 1906, and a Member on February 28, 1911.

WALTER GLADDEN HUNTER, M. Am. Soc. C. E.*

DIED APRIL 1, 1923.

Walter Gladden Hunter, the son of William J. and Emma L. Hunter, was born on April 19, 1879, at Arcata, Calif.

His early education was obtained in the different towns along the line of the Northwestern Pacific Railroad with which Company his father was connected. In 1897, he entered the University of California where he received his technical training, and from which he was graduated in 1901.

In 1901 and 1902, Mr. Hunter was employed as Draftsman and Engineer at the Holt Manufacturing Company at Stockton, Calif., and from 1903 to 1906, he was in the Federal Service in the Philippine Islands as Assistant Engineer in the Bureau of Engineering and Public Works. In this position, he had charge of the construction of highways and the investigation of river-control and irrigation projects. From 1906 to 1908, he was a member of the firm of Hunter and Hunter, at Tonopah, Nev., and was interested in mining and highway work.

Subsequently, Mr. Hunter engaged in private practice in California, Arizona, and Nevada. For five years, he was associated with Robert M. Morton, Assoc. M. Am. Soc. C. E., now State Highway Engineer of California, under the firm name of Hunter and Morton. From 1912 to 1914, as City Engineer of Stockton, he designed a new sewerage system and many other important structures.

In 1913, Mr. Hunter was engaged in engineering work in connection with highway construction, for Sonoma County, California. He also acted as Engineer for the Stockton and Mokelumne Canal Company (supplying 32 000 acres). In 1914, he served as Construction Engineer for Yuma County, Arizona, planning the expenditure of $500 000 for highways, and also as

* Memoir prepared by George W. Leistner, Esq., Stockton, Calif.

Engineer for the West Side Irrigation District and for Reclamation District No. 684, both in San Joaquin County, California.

Later, Mr. Hunter acted as Engineer in an advisory capacity to the City of Stockton. In 1918, he enlisted in the U. S. Army, receiving a commission as Captain, and was with the troops that were embarking from New York City on the day the Armistice was signed.

During the last two years of his life, Mr. Hunter devoted much time to work at the Stockton Golf and Country Club. An ardent and expert golfer, he loved his home course and made it a place of beauty, and it was during a game on this course that he met his death by lightning.

On November 21, 1906, Mr. Hunter was married to Miss Agnes Ferguson, of Stockton, who survives him. He is also survived by his father who is Superintendent of the Northwestern Pacific Railroad Company at Eureka, Calif.

Walter Hunter was loved by those who knew him. Modest and retiring, he was a man of public spirit, always striving to serve his fellow men.

Mr. Hunter was elected an Associate Member of the American Society of Civil Engineers on April 30, 1912, and a Member on May 31, 1916.

GEORGE ALBERT KIMBALL, M. Am. Soc. C. E.*

DIED DECEMBER 3, 1912.

George Albert Kimball was born on May 14, 1850, at Littleton, Mass., the son of William and Mary A. (Lawrence) Kimball. His earliest known ancestor in America was Richard Kimball. His great-grandfather, Daniel Kimball, was a First Lieutenant during the War of the Revolution.

Mr. Kimball's education was obtained in the public schools of Littleton and at Appleton Academy, New Ipswich, N. H., where he prepared to enter Dartmouth College and the Thayer School of Civil Engineering. He was obliged to relinquish his studies, however, on account of eye trouble.

Mr. Kimball's early life on the farm at his home in Littleton gave him that love of Nature which was evident throughout his life. His business education probably began in the typical country store, in Littleton, owned by his father, who also served the town as Selectman and Town Clerk.

In later life, Mr. Kimball was fond of relating his experiences in the store and in trading the products of the farm near his home and in and around Boston, Mass., in which occupation, he made many life-long friends among his customers.

He began his chosen profession in 1869 in the office of Frost Brothers, Engineers, in Somerville, Mass., as Rodman on railroad surveys and, in the following year, as Transitman on city work. He continued with this firm until 1874, when he opened his own office in Somerville. In 1876, he was elected City Engineer and continued in this position until 1887. From 1880 to 1887, he served the city as a member of its Board of Health. In 1887, he

* Memoir prepared by J. P. Snow and J. R. Worcester, Members, Am. Soc. C. E.

resigned his position as City Engineer of Somerville and opened an office in Boston. From 1891 to 1900, he served as a member of the Somerville Water Board and as an Alderman from 1888 to 1889.

In 1888, Mr. Kimball was made a member of the Massachusetts Grade Crossing Commission which conducted an extensive investigation and reported on all the grade crossings in the State. In 1896, he was appointed by Governor Roger Walcott to the Metropolitan Sewer Commission, a position which he held until 1901. In his work in Somerville, he early had established a reputation as an expert in sewer design and construction and in sewage disposal, and although his later life was largely devoted to entirely different work, he continued until his death as Consulting Engineer on Sewers for the Massachusetts towns and cities of Millis, Malden, Brockton, Milford, New Bedford, Winthrop, and Peabody, as well as of Montpelier, Vt.

In 1896, Mr. Kimball was appointed Chief Engineer of the Boston Elevated Railway Company which position he held until his sudden death in 1912. It was at this time, perhaps, that he became best known in his profession, and to this work he gave the crowning efforts of his life. This work included the construction of all the elevated railway structures in and around Boston, as well as the Cambridge Subway and Tunnel. This last named work received his particular attention, and it is probable that there was no one piece of construction or achievement of his, which gave him more satisfaction or in which he took more pride than this. During its construction, rules for assuming safe units for the bearing power and the pressure of various classes of earth were developed under his direction for different stages of compaction as newly deposited, well packed, or undisturbed.

In 1902, following a severe illness, Mr. Kimball was sent by the Company to visit Europe and study the systems of rapid transit in the great cities of England, Scotland, France, Germany, and Austria.

In 1911, he attended the excursion of the Society to the Panama Canal, and had charge of the Eastern party. In conjunction with George G. Anderson, M. Am. Soc. C. E., who had charge of the Western party, he published an interesting pamphlet describing the work as it existed at that time.

Mr. Kimball gained prominence in his profession through hard, conscientious, and constant work during all the years of his practice. These attributes, coupled with the sterling qualities of enterprise, honesty, and fair dealing, which he possessed in an eminent degree, brought him prominently before public men of influence.

It was always his aim to secure the very best and most effective work, both in the methods used and in the construction itself. He was not too closely bound by precedent, yet he followed what was known to be the best practice. Conservative in judgment, he was open minded toward any new idea or method, and if he felt its value had been confirmed by theory and experience, he was not slow to make use of it.

The great number of Mr. Kimball's friends in every walk of life bear testimony to the high esteem in which he was held. His unfailing courtesy, fair dealing, and kindness, together with a remarkable control of himself under the

most trying conditions, fitted him to carry through his difficult tasks. He was never hasty to reach conclusions. His judicial temperament led him to weigh conflicting opinions with great care. He was quick to sift the important from the irrelevant and to give proper weight to the essential things. His unfailing sense of humor smoothed over any bitterness between disputants. He had a habit of introducing conferences on delicate questions by boldly attacking and laying bare the heart of the trouble, and then searching for the redeeming features, which always left every one in good humor. Although not much given to public speaking, he was a faithful member of the organizations to which he belonged, and when he took part in discussions, his contributions were always valuable. He was a clear-headed, careful witness in expert cases, and his testimony carried conviction to all who heard it.

Mr. Kimball was married in 1872 to Miss Lizzie E. Robbins, who, with four children, two sons and two daughters, survives him.

At the time of his death, Mr. Kimball was a member and Past-President of the Boston Society of Civil Engineers, a member of the Institution of Civil Engineers, the American Institute of Consulting Engineers, the New England Water Works Association, and the Street Railway Club, as well as of several fraternal and social clubs.

Mr. Kimball was elected a Junior of the American Society of Civil Engineers on May 12, 1875, and a Member on July 1, 1891. He was a member of the Board of Direction of the Society at the time of his death.

EDWARD THOMAS EVERY MILLER, M. Am. Soc. C. E.*

DIED APRIL 17, 1923.

Edward Thomas Every Miller, the eldest son of the late Colonel Miller who served in the British Army in India, was born on October 25, 1873, at Boulogne, France. He was educated at Framlingham College, Suffolk, England.

In 1892, Mr. Miller received an appointment as Supervisor of the 12th Division of the Assam-Bengal Railway in India, his work consisting of the construction, leveling, estimating, and measuring of earthwork. In 1896, he was appointed Personal Assistant to Superintending Engineer R. S. Routh of the Third Section of that railway, remaining in this position for three years. During this time, he was engaged in drafting and estimating, and also had charge of a sub-division (10 miles) of the Hill Section, including tunnel construction. In 1899, he transferred to the Traffic Manager's Staff, with which he remained until 1901, when he left India for England on sick leave. Acting on medical advice, Mr. Miller never returned to India.

In 1902, he went to the Argentine Republic, where he accepted a position as Assistant in the General Manager's Office, of the Buenos Aires Great Southern Railway, in which capacity he was engaged with technical matters of train running and new capital works. In 1906, he was appointed Sectional Engineer on the construction of the Central Cordoba Railway Extension (100 km.) to Buenos Aires.

* Memoir prepared from information on file at the Headquarters of the Society.

In 1908, Mr. Miller became Assistant Sectional Engineer of the Headquarters Section of the Central Argentine Railway, and in the same year was appointed Sectional Engineer of the Third Section at Galvez, where he had charge of final construction on the Morteros-Hersilia Extension (72 km.), including the building of three stations. In 1910, as Sectional Engineer of the Cordoba Section (700 km.), he was given full charge of all new works. He retained this position until 1915, when he was appointed Sectional Engineer of the Rosario Section, in charge of two sets of railway workshops, moles, docks, etc.

From 1920 until his death, Mr. Miller held the position of Assistant to the Chief Engineer of the Central Argentine Railway, with headquarters at Buenos Aires.

He died of heart failure on April 17, 1923, and is survived by his widow, a son, and a daughter.

Mr. Miller was elected an Associate Member of the American Society of Civil Engineers on December 5, 1911, and a Member on October 11, 1920.